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MANUAL

FOR

MINOR IRRIGATION WORKS

Issued by :

COMMUNITY PROJECTS ADMINISTRATION
GOVERNMENT OF INDIA
MARCH, 1956

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OFFICE OF THE PRINCIPAL, GRAMA SEVAKS' TRAINING CENTRE
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PREFACE

This Manual for Minor Irrigation Works has been prepared for the use and guidance of officers working in the Community Projects. It is not intended either to write a thesis or to produce any comprehensive and highly technical book on irrigation. What is aimed at is to collect and produce some material which would equip the staff working in the Community Projects with the basic knowledge of the principles of irrigation engineering and practice. For designing any major irrigation work, use of more detailed treatise would of course, be necessary. In spite of best efforts to make the Manual fit for popular reading, it has not been possible to avoid being technical in many places. The idea is, that with this Manual in hand, the field staff should be able to proceed on correct lines and be in a position to undertake independently the survey, investigation and design of a minor irrigation work. Technical matter has been mostly collected, summarised or extracted from various standard books on engineering and references have been quoted in the foot-note in each case. A bibliography showing the books of reference appears at the end.

2. The Draft Manual was circulated to Chief Engineers, Irrigation, of several States and their comments are gratefully acknowledged. These have been extremely useful in introducing suitable modifications in the original draft. Since conditions in one part of the country differ very widely from those in the other, it is needless to add that any attempt to lay down a definite set of rules and specifications to be followed all over the country would be futile. A local note indicating the data actually in use in any particular part of the country will have to be appended to the Manual.

CHAPTER 1

INTRODUCTION

Never before, in the history of development of irrigation in this country, has Government's attention been so much focussed on the construction and improvement of minor irrigation works as it is to-day. The decreasing productivity of land, closing down of imports from Burma, separation of the most fertile tracts of Punjab due to partition, the ever-increasing population and a whole host of other causes have resulted in an acute food shortage and the food problem in our country has assumed stupendous proportions. Increased food production through a large-scale intensive agricultural programme has become the crying need of the day and irrigation occupies the most important place in this programme.

Of the total expenditure visualized in the Five Year Plan, irrigation and power have claimed 30 per cent. A significant feature of the irrigation part of this programme is the stress laid on Minor Irrigation Works. These works are capable of giving quick results and are sure to cut down substantially, the large yearly imports in the shortest possible time. Though this has been one of the main considerations for their inclusion in the Five Year Plan, there is a more important aspect of these works. Irrigation in India is vital to its rural development but rural development is now recognised to be a human problem—this is the message of the Community Projects. There can be no two opinions about the great utility of the major irrigation works planned and under execution all over the country, and their contribution to the increase in country's food production would undoubtedly be very great. But minor irrigation works is a people's programme—there is a touch of human approach towards the Indian peasant in this programme—a feature which is hard to achieve in the major irrigation works. It is against this background that we have to judge these short-term schemes designed and planned not only to give immediate results but also to bring about a change in the outlook of the Indian farmer.

The minor irrigation works in this country have not received the attention of the engineers in the past that has probably been due to them and in the pages that follow, an attempt has been made to touch the various aspects of design and operation of such works

CHAPTER II

IRRIGATION THROUGH THE AGES

Irrigation in India has been practised since times immemorial but the story of development of irrigation during the past few centuries and the magnificent irrigation works built during this period read like a legend. It is the story of some of the most brilliant feats of engineering ever performed by mankind, which have turned vast areas of desert and partially cultivated land into productive and extremely fertile tracts. Undoubtedly, these works have been a striking success from a commercial viewpoint, bringing sometimes huge revenues to the State exchequer, but by their very nature these are essentially welfare works with which has been linked the fate of starving millions in this country.

Even remnants of some old irrigation works speak of the extraordinary ability and technical skill displayed by the engineers of olden times. The earliest evidence is traceable in the Vedas written some 5,000 years back in which mention has been made of canals, wells and dams. During the past millenniums, irrigation received special attention under the rule of Hindu kings of South India, who got irrigation works constructed mostly for humanitarian reasons. Ruins of ancient irrigation works are found scattered all over the country and many of them are still in use. The Sudarshana Lake (300 B.C.—457 A.D.) near Kathiawar constructed and perfected during the reign of the Maurian Emperors Chandragupta and Asoka and finally destroyed after a chequered existence of over 700 years, the marvellous Bhojpur lake (11th century A.D.) near Bhopal, the largest and most beautiful sheet of fresh water covering an area of 250 sq. miles, constructed across the Betwa valley and fed by the Kaliasot river by the construction of a magnificent dam on which stands the old fort of Bhopal, thus draining additional 500 sq. miles of catchment into the lake, the old Jamuna canal or Ferozeshah Tughlaq canal (end of the 14th century), a description of which appears later and many more, which it is beyond the scope of this Manual to narrate—these are some of the magnificent works constructed by our ancient rulers, which bear testimony to the high technical skill and practical ability of the Indian engineers of the time.

The conditions in Northern, Central and Southern India vary so considerably that the development of irrigation in these parts during the last few centuries is a story by itself.

1. **Irrigation in Northern India***.—The development of irrigation in Northern India dates back to the 14th century when Feroze Shah Tughlaq constructed a 150 mile length of canal taking off from the right bank of the Jamuna river, to irrigate his gardens at Hissar. Gradually, this canal fell into a state of disrepair till it was restored by Akbar during whose period it came to be known by the name of 'Nehr-i-Shahbab'. Later on, the Shahjehan branch canal taking off from the main canal was constructed to bring the canal water to the imperial palace and gardens at Delhi. A few other canals were also constructed during the Mughal period, but with the collapse of the Mughal dynasty, the conditions of these canals deteriorated and eventually these went out of use completely.

It was in 1817 when for the first time, the attention of the British Government was drawn to the restoration of these canals. After necessary repairs, the Shahjehan branch canal was re-opened and water from there reached Delhi again after a lapse of 67 years. During the next 20 years from 1823 onwards, the main canal to Hissar was revived, other branches re-modelled, some extended and some reconstructed and the whole system, which is today known as the "Western Jamuna Canal" came into full operation. With this is associated the name of Sir Colvin, who is aptly known as the 'Father of Irrigation' in Northern India.

For want of any previous experience in irrigation engineering, many mistakes were made and the canal system suffered from several defects. Experience gained in the Western Jamuna Canal system was, however, fully made use of in the planning of the Ganges Canal Project. Sir Cautley of the late Bengal Artillery figured most prominently in the design and construction of this canal system. The first investigations for the site of head-works were made in the year 1836 near Hardwar. During 1837-38, there was an acute famine and this accelerated the progress on investigations which resulted in the finalization of the site for head-works at Kankhal. The work of construction was taken in hand, the same year. Soon after, the country was involved in the First Afghan War and Government were forced to stop or at least slow down work on all major undertakings. Apart from this, the canal irrigation was being introduced for the first time and doubts were entertained as to the stability of the Solani Aqueduct. It was feared that it might get damaged during an earthquake. Also, consequent on the diversion of the river water into the canal, it was apprehended that navigation in the Ganges river may suffer. In addition, there was the danger that the presence of the canal might cause large scale water-logging giving rise to the spread of malaria. Government also considered the desirability of

*Summarized from the Military Engineer in India by Lt. Col. E.W.C. Sandes.

using the proposed Ganges canal purely as a navigation system. All this created lots of difficulties and for a time, it appeared that the whole project may have to be abandoned. In 1847, however, the navigation scheme was dropped and simultaneously the work on construction of the Ganges Canal Project as originally prepared re-started on a more vigorous scale. By 1848, the Solani Aqueduct was completed and in April of the same year, the Ganges canal was opened.

The works built in the first 20 miles of the Ganges canal from Hardwar to Roorkee speak volumes of the skill, energy and perseverance of the great genius behind this gigantic project. The first 12 miles of the canal are in heavy cutting and in the 6th mile, it crosses the Rani Rao torrent, transported overhead in a masonry aqueduct termed as 'Super-passage'. The Ratmau torrent in the 13th mile at Dhanauri offers an equally astounding spectacle. The river flows straight into the canal. In times of flood, the entry of the river water into the canal is controlled by means of a regulator and the river water is allowed to escape over a weir. When the floods subside and the canal has to be run, the regulator gates are opened and the canal water passes into it freely. In the 19th mile is the Solani Aqueduct which has a clear water-way of 750 ft. provided in 15 arches which carry above it the main canal 164 ft. in bed width. All these rest on foundations 250 ft. wide with wells sunk 20 ft. below ground level. The construction of the Ganges canal is a brilliant feat of engineering and has laid the foundations of one of the most elaborate and remunerative canal systems in the whole world. It is said that during the famine of 1865-66, this canal repaid to the country, more than its capital cost.

The Ganges canal had, however, many defects and was subject to severe criticism by the engineers of the later days who in fact, opined that the whole system would need a complete remodelling. However, nothing much was done to the existing works but to rectify the defects in this canal system, another head-works was constructed at Narora in the year 1886 from which takes off the lower Ganges canal. This irrigates the lower portion of the Ganges Jamuna doab, not previously served by the original canal. The lower Ganges canal itself had a chequered career as it suffered from a terrible disaster in 1885 when the aqueduct over Kalinadi was destroyed by an unprecedented flood. The structure which was subsequently constructed to replace this is even bigger than the Solani Aqueduct.

This, in brief, is the story of the Ganges and Jamuna canals, which later on grew into an extensive irrigation system in the whole of the Northern India. The Ken, Betwa and Sarda canal system in the U.P., the Bari Doab canal, the Sirhind canal and Lower Chenab

canal in the Punjab—all these are magnificent irrigation systems, each having a history of its own, which it is not within the scope of this Manual to narrate. The Sirhind canal includes the Bundki Super-passage, the largest work of its kind in India and is a monument to the ability of its designers and builders, who have profited by the lessons of the Ganges and the Bari Doab canals. During the later part of the 19th century, irrigation extended farther and farther into the North West reclaiming desert after desert. The lower Chenab canal can claim with considerable justification to be the most extensive and successful irrigation system in India and probably in the world, yielding an annual return of more than 45 per cent. on its capital account.

Even though Sind is not now a part of India, the story of development of irrigation in that desert province forms a part of the story of irrigation in Northern India and a brief description of this is necessary to complete the story. The inhabitants of Sind had tapped the water of Indus by scores of small inundation canals which silted up every year, and required frequent re-excavation. The system was unsatisfactory and costly. It supplied water only to tracts near the river and when that water was most needed, it was not available. All these canals, which were only natural flood spills of the Indus were adapted for irrigation in the later part of the 19th century. The irrigation engineers of the time persistently advocated the construction of perennial high level canals for Sind and their efforts bore fruits in the gigantic Sukkur Barrage, nearly a mile in length, which spans the Indus and supplies 7 perennial canals. The "Sukkur Barrage and Canals" is one of the greatest irrigation undertakings which the world has yet seen. Begun in July, 1923, it was opened by Lord Wellingdon in 1932 and irrigates 7½ million acres of land.

The irrigation works in Bengal and Orissa, which form the remaining parts of the Northern India, are of not any material importance and do not need special mention. In fact, these parts have a normal rainfall which in some districts is so copious that irrigation may even be harmful.

2. **Irrigation in South India***.—Conditions in South India differ fundamentally from those in the Northern India. In the latter regions, the entire canal system is fed by a net-work of perennial rivers which have their source in the snow clad Himalayan mountains—Nature's unlimited source of water supply. During the whole winter season, the rivers carry enormous quantities of water and the canals run practically full, making irrigation of *rabi* crops comparatively an easy task. In the rainy season, of course, the rivers carry much bigger quantities of water and no difficulty is experienced

*Summarized from the Military Engineer in India by Lt. Col. E .W. C. Sandes.

whenever it becomes necessary to open the canals. On the other hand, in South India, all the rivers are rain-fed. During rains, these rivers swell into huge streams, but after rains the flow in them dwindles down to a trickle. Practically, no water is left in these rivers to do irrigation of winter crops, which have therefore, to depend entirely on the North-East Monsoon. Irrigation practices here are, therefore, necessarily different from those in the Northern India and works now under operation mostly consist of irrigation tanks at the head of river valleys often linked with anicuts or pick-up weirs lower down.

Due to uncertain and capricious nature of the North-East Monsoon, several tracts in South India used to be the scene of frequent famines and failure of crops came to be regarded as Nature's calamity. It was in the year 1804, when serious concern was shown for the first time by the British engineers, to control the Cauvery river for protecting the delta district of Tanjore which was threatened with ruin. Irrigation in the Cauvery Delta System is of great antiquity dating back to the 2nd century B.C. The island of Sringapatam situated 20 miles up-stream of the head of the delta, divides the river into two branches—the Coleroon and the Cauvery. The Coleroon carried the flood waters and the Cauvery was meant to carry irrigation water to the delta, where it used to irrigate several lakh acres of land. In the early part of the 19th century, it was noticed that there was considerable retrogression in the Coleroon which offered a more direct course to the river than the Cauvery. The result was that most of the water escaped into the Coleroon and Cauvery could draw very little of it. This threatened the irrigation of the whole Delta System and it was clear that, within a few years, unless Coleroon was controlled, practically no water would be available in the Cauvery for irrigation of the Tanjore district.

It is at this stage, that Sir Arthur Cotton, another brilliant engineer, to whose credit stand, some of the most magnificent works in South India, comes on the scene. He clearly saw that unless the rapacity of Coleroon was brought under control, there would be no chances of any supplies being diverted to the Cauvery; and his investigations resulted in the construction of a big barrage in 1834, known as the "Upper Anicut". The barrage, provided with several under-sluices, was constructed across the Coleroon at its head and was essentially meant to divert requisite supplies to the Cauvery. In the earlier stages of irrigation, only the flow-water of the river had been utilized but by constructing the "Upper Anicut" of sufficient height, Cotton managed to impound a big storage and was able to give large and assured supplies in winter, whenever the North-East Monsoons failed.

The construction of this barrage is one of the biggest and most daring undertakings in South India and for a time, it changed the whole face of the Cauvery delta. Soon, however, it was discovered that Coleroon was getting silted up rapidly, its bed almost reaching the crest of the anicut and the main river was carving out a deep course through the Cauvery which threatened to become the main flood carrier in place of the Coleroon. The danger of the whole of the Tanjore district getting flooded with waters of the Cauvery seemed imminent and the need to protect the district from this impending disaster was urgently realized.

Further investigations resulted in the construction of Cauvery dam and drastic re-modelling of the upper anicut in 1843-45. The Cauvery dam consists of a masonry bar at the head of the Cauvery river close to Sringapatam. The crest of the "Upper Anicut" was lowered and it was fitted with lift shutters; the ventage in the scouring sluices was also considerably increased. With all these improvements and re-modelling, although the work as originally constructed by Cotton, changed beyond recognition, yet the dash, the keenness and the high technical ability he displayed in initiating the works in the Cauvery delta inspired such confidence that Government were encouraged to undertake similar works in the Godavari and Krishna deltas also.

The Godavari river after it emerges out of the Eastern Ghats, enters the coastal plains at Polavaram, as a broad and shallow river. Just below Rajahmundry, it divides itself into two main branches—Gautami and Bashishta which enclose the central delta. Each of these two main branches sub-divides itself and throws its water into the sea, through a number of smaller streams which constitute the East and West Godavari deltas. Recurrence of famines due to failure of monsoons had become a common feature in the Godavari district and almost immediately after the great improvements in the Cauvery Delta System were completed, Cotton was again called upon to harness the big Godavari river for the benefit of the Godavari peasants. The scheme of the Godavari Anicut System was prepared and sanctioned almost immediately and the work constructed under his guidance. The anicut was built in 4 sections linking the island in the broad river and the head works consisted of a head-sluice and an under-sluice for each of the three deltas. The completion of the Godavari project opened a new era of prosperity and plenty in this famine-stricken district. "To those who have not the local knowledge the fact that the value of the crops now grown annually in this delta is several crores, will convey a very meagre idea of the benefits of irrigation. The Godavari ryots have recently erected at Dowlaishwaram, Bobberlanka and Chettipetta, statues to

commemorate the memory of Sir Arthur Cotton, originator of the scheme and by doing so have shown their appreciation of what he has done for the district by his monumental works for securing copious supply of precious water to their land"*. The central delta, particularly the "Nagaram" island is known as the "Garden of the Godavari district" and is noted for the fine Gannavaram Aqueduct—a remarkable feat of engineering. It was constructed in 1852 and carries one of the central delta canals over a branch of Godavari and consists of 40 spans each of 45 ft. water-way carrying 2 channels 24 ft. wide. All the canals in the 3 deltas are also navigable throughout their length for 11 months of the year, and this has helped a great deal in the export and import trade of the tract. Like the Cauvery Delta System, here also lots of re-modelling work had to be done, and the anicut had to be slightly raised and fitted up with collapsible shutters which fall automatically during floods.

Simultaneously, proposals had been made for supplying irrigation to the delta of the Krishna—the neighbour of Godavari—which had also been badly hit in the great famine of 1832-33. This project was also sanctioned in 1851 and the Krishna, was dammed at Bezwada. At the site chosen for the Krishna weir, the river flowed between rocky hills, 3,900 ft. apart, but although this width was much less than at Dowalaishwaram on the Godavari, it required the ability and skill of the engineers to build on pure sand a masonry dam 15 ft. high across a gorge in which the river rose 40 ft. when in flood. In 1855 the work was successfully completed and thus brought the blessings of irrigation to the Krishna delta also.

In the years that followed, many other anicut systems were built and developed bringing the benefits of irrigation to one delta after another. Soon, however, a time reached, when it was realized that the normal flow of most of the rivers had been fully tapped and additional supplies could be had only by the construction of storage reservoirs across the river valleys, to impound the rain water which had hitherto been running waste into the sea without conferring any benefits on the land through which it passed. The name of Penny Cuick, another brilliant engineer, will ever be associated with the construction of the Periyar dam one of the boldest irrigation projects in India. The main feature of the Periyar System is the diversion across the peninsula into the Bay of Bengal, of a large river which would naturally flow into the Arabian sea. To achieve this, Penny Cuick built a masonry dam across an inaccessible gorge in thick malarious jungle. The dam is of concrete with masonry facing 170 ft. in height above the river bed, and 1,241 ft. long. Water from this

*Public Works Department, Madras State—Irrigation Administration Report—1950-51

big lake is carried into a tunnel 5,700 ft. long bored through the ridge and is dropped into a tributary of the Vaigai river. After irrigating a portion of the land in this feeder valley, it is again picked up by a regulator across the Vaigai 80 miles down stream of the dam and irrigates the thirsty land of Madurai. The ravages of the floods passing through the narrow gorge, the extremely malarious conditions of the tract, the lethargy of labourers through sickness and the complete lack of any kind of training—how amidst all these difficulties this genius of an engineer completed the construction of a work of such stupendous magnitude, baffles one's imagination. The difference that this scheme has made to the prosperity of the Madurai district is enormous.

After the completion of the Periyar reservoir project, many other tank schemes were investigated. By the very nature of their construction, these schemes were bound to be much more costly than the anicut schemes in the deltas and with the irrigation water rates prevalent in those days such schemes could not pay for themselves except when the project was very big. Most of the tank schemes were, therefore, abandoned. Only in 1929, the construction of Stanley reservoir at Mettur was started and completed in 1934. The Cauvery Mettur project is intended to ensure supply in the delta district and to extend irrigation in another three lakh acres of land. Mettur dam, 214 ft. high and 5,300 ft. long is one of the biggest masonary dams in the world. Its water spread area is 60 sq. miles and it impounds 93,500 million cubic ft. of water. The reservoir also acts as a flood moderator and protects the Tanjore district from the ravages of the Cauvery when it swells up during the South-West Monsoons.

In the Bombay presidency the strip of mountaneous coast country interspersed with high plains covered with friable black soil needs little or no watering. Elsewhere, if irrigation is contemplated, it is only by means of reservoir. The Khadakwasla reservoir (still known by the name of its builders as Lake Fife) 5 sq. miles in area, the Bhandardara dam, 270 ft. in height south of Nasik, and the Bhatagar dam—these are some of the famous works of Bombay presidency.

3. **Irrigation in Central India***.—The story of irrigation works in Central India, specially in Deccan is so closely linked with the history of its people, the rise and fall of various kingdoms and above all, the growth of its civilisation, that in the remains and ruins of its old lakes, one sees the Deccan history unfolding itself. The countless sheets of water surrounding the 30 ruined cities recently excavated by the archæologists can still be seen from the hills of Panigiri in Nalgonda district. The ancient rulers showed great ingenuity in the construction of these lakes and the tradition was kept up by the

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* Summarised from "Projects for Prosperity" by Sadath Ali Khan.

later kings like the Chalukyas, the Rashtrakutas, the Kakatiyas and the Yadavas who built innumerable works each during his own time.

Telingana with its 7 districts is aptly known as the "Land of Thousand Tank". Of the 25,000 tanks in the whole State of Hyderabad, 80 per cent. are located in Telingana. There are several big lakes like Pakhal, Ramappa, Lakhnavaram and Ghanpur belonging to the 12th and 13th centuries. Close to these lakes are situated several temples of great architectural beauty around which the civilisation of this ancient land grew. The Pakhal lake constructed in 1230 A.D. was breached by Shitab Khan, the commander of Delhi Emperor's forces. Ramappa lake close to Warangal is another magnificent work, where exists the famous Ramappa temple. There are many more tanks of historical importance constructed all over Telingana.

Later on the Muslim rulers came. Their highly developed aesthetic sense was reflected in almost all the works they built. With the help of imported architects from Paris, they introduced several features of architectural beauty in the irrigation works also. The huge dam Khatlug Talao, built by Mohammad Bin Tughlak, the Hussain Sagar lying between the twin cities of Hyderabad and Secunderabad, and the Mir Alam Tank, the only "Multi-Arch Dam" of its kind in the world, are some of the famous works built by the Muslim rulers.

At the close of the last century and the beginning of the present one, Deccan faced some of the worst famines. The years that followed saw intense activity and schemes were formulated to harness the river waters in the manner which was most economical and beneficial to the people. Most of the major and minor projects now in operation, were conceived during this period. With all these works are associated the names of Sir Visvesvaraya and the late Nawab Ali Nawaz Jung, both eminent engineers of South India. The Usman Sagar, the Himayat Sagar, Pocharam, Wyra and the Palair projects were the first to come up. Among the major projects, the Nizamsagar reservoir is one of the biggest irrigation tanks in the country, planned and constructed under the personal direction of Nawab Ali Nawaz Jung.

Further South in the Mysore plateau is the Krishnarajsagar reservoir behind which are laid the world famous "Brindavan Gardens". In the words of Sir M. Visvesvaraya, who developed this project, "the scheme was to open out a vista of possibilities of ever increasing value in the State".

Next only in size to Mattur dam, this reservoir holds 48,000 million cubic ft. of water and irrigates 1,25,000 acres of land in the Mandya District.

Storage works made an early appearance in dry and dusty Rajputana, where there are no rivers on which reliance can be placed for irrigation. Some very fine irrigation reservoirs have been constructed in Jaipur and Ajmer Merwara where the name of Col. Dixon is still gratefully remembered for the numerous irrigation tanks he got constructed. The latest territories in India to receive the boon of irrigation were the central provinces (now known as Madhya Pradesh). Irrigation on a small scale had been practised by the people of these parts for centuries but extensive works were not considered necessary until a cycle of dry years beginning in 1897 culminated in a disastrous famine in 1899. Several irrigation projects were investigated at that time. There are no snow-fed rivers in the country around Nagpur, Jubbulpore and Raipur and the largest rivers such as the Mahanadi and the Wainganga (tributary of the Mahanadi) carry little water except during monsoons between June and October. Reliance had accordingly to be placed almost entirely on reservoirs, the only attempt at irrigation by direct flow being made on the Wainganga and Mahanadi. In these parts the scarcity of rains in September and October causes failure of crops and the reservoirs are primarily intended to give irrigation water during these months. There are now three canal systems in these provinces —the Wainganga near Balaghat, the Tandula, close to Durg, and the Mahanadi close to Raipur. These are reinforced by some two dozen large reservoirs and more than 100 small tanks.

CHAPTER III

WATER REQUIREMENTS OF CROPS

"More water better crops" is a very common, though mistaken, belief among the cultivators. With the increasing demand of irrigation water, it is essential that the cultivators understood the optimum water requirements of crops and thus made maximum use of the available water. It is only by judicious irrigation, combined with a knowledge of the water requirements of crops at various stages of their growth that this objective can be achieved.

In irrigation engineering the term "duty" is used to indicate the water requirements for various crops. The outlet duty means the **area irrigated per cusec of water issued at the outlet.** "Cusec" is a term used to denote the rate of flow of water. **One cusec means, one cubic foot of water flowing per second and is equal to about 22,000 gallons flowing per hour. The "base" of a "duty" is the time during which the flow is continued.** If one cusec running continuously for four months will mature 100 acres of crop, the duty in that case is said to be 100 acres to the cusec to the base of four months. **Delta ($\triangle$) is defined to be the depth of water in feet required to mature a crop.** If B be the base and D the duty of water, delta ($\triangle$) would be given by

$$\Delta \times D \times 3560 = B \times 24 \times 3600$$

or

$$\Delta = 2 \frac{B}{D} \text{ approximately.}$$

Similarly, the canal duty means the area irrigated per cusec of water issued at the sluice gate. For storage reservoirs **the "tank duty" is used to mean the area irrigated per million cu. ft. of the tank storage.**

The requirement of irrigation water for various crops depends upon many factors*. These are:—

(i) Climate

(a) Rainfall, its incidence and distribution.

(b) Temperature.

(c) Wind movement.

(d) Soil evaporation.

(e) Relative humidity.

* From proceedings of The First Meeting of the Standing Advisory Committee on Water Requirements of Crops.

(ii) Soil texture as determining its retentivity.

(iii) Drainage conditions.

(iv) Sub-soil water table.

(v) Use of different kinds of manures.

(vi) Crop rotation.

(vii) Method and frequency of irrigation.

(viii) Quality of water both physical and chemical contents.

(ix) Method of delivery viz. volumetric or area irrigated basis.

(x) System of irrigation of fields.

(xi) Cost of water; and

a whole host of other factors.

The conservation of soil moisture is one of the most important factors affecting the water requirements of a crop. Thus deep ploughing helps the retention of moisture and promotes economy in its use. Similarly, it has been observed that there is a definite economy in the water requirement of the plant by the application of farm yard manure.

Experiments have been performed by the Agricultural Department in various parts of the country on the water requirements under varying climatic conditions. The results obtained under laboratory conditions, while they offer valuable guidance in planning the cropping programme cannot be taken to indicate the actual water requirements under the field conditions. The profile characteristic of natural soil is different from the picked up soil in the pot culture experiments. The method of irrigation i.e. surface irrigation in the field and sub-irrigation in the laboratory, wide fluctuations in the soil moisture conditions in the field, contribution of the sub-soil water etc. etc.—these are some of the factors which give rise to wide differences between the results obtained in the laboratory and those in the field.

The requirements of water are also governed by the economic conditions of the people, comparative availability of land and water, and various other factors. In dry provinces where water is available in a very limited quantity, the attempt is to obtain the best dividend from the cusec of water rather than from the acre of land and the result is increased duty. An important economic factor affecting the water requirement is the land holding and the labour available per unit of this land. If a cultivator has a small holding and he has surplus of labour i.e., the number of persons to be maintained is big (this would ordinarily consist of members of a family), a given quantity of water would be distributed on as large a plot of land as

possible. The use of water would thus be greatly economised resulting in increased duty. This holds good for water lifted from a well. On the other hand, if there is a big land holder who does not have the necessary labour at his disposal for managing the proper distribution of water, but can get ample quantity of canal water by paying for it, he would have a tendency to use comparatively much more water. The result would be a lower duty. The water requirements of the main crops, for which irrigation water is generally required are given in the following paragraphs*.

Rice.—Pot culture experiments made in the agricultural research stations to determine the exact water requirements of the rice plant on dry weight basis show that the actual loss of water by transpiration during the complete life cycle of the plant is about 25″ and is 27·4″ inclusive of evaporation. In these experiments a normal growth and yield of crop could be obtained by maintaining the moisture conditions at 20 to 21 per cent. of the oven dried soil. It would thus seem doubtful whether rice really requires those extremely wet conditions as is believed by the cultivator. Investigations also show that flooding the field after transplantation for about 3 weeks followed by subsequent de-watering is beneficial to the plant. This temporary submergence is specially beneficial to the growth of tillers. Later on, however, a field with sufficient moisture but no water standing in it favours tillering, while water definitely suppresses it. The standing water, however, increases the height of the plant and does not seem to affect the plant adversely. It is believed that water produces and maintains a certain texture and temperature condition of the soil suitable for young seedling during the earlier stages of growth. It is also believed that some soils provide certain constituents which are not available in dry soil and hence the need for submergence. On the other hand, pot culture experiments made with the heavy soil of Madhya Pradesh show that rice grows better when not submerged with water.

A great deal of field work has been done by the Agriculture and Irrigation Departments in order to determine the duty of water for rice. In Madras because of the big differences in soil it is as low as 50 acres in Coimbatore, about 70 in Tanjore and nearly a 100 in Godavari. In Sind, experiments conducted with different duties have shown that a duty of 45 was too little, 35 to 40 produces a good crop, while 30 and under was found to be necessarily excessive. In Bengal, the amount of 40″ total depth is held to be the minimum necessary for the successful growth of crop. As regards the periodic water requirements, it is supposed that there are three marked periods of

*Extracted from Board of Agriculture and animal Husbandry—A review of the work done on Water Requirements of Crops and an Appreciation of the present position with suggestions for the future.

high water requirements, the first during the seedling stage covering the period of about ten days, the second during the pre-ploughing stage covering about 25 days and the third during the period when the grounds are ploughed covering the space of 5 to 7 days.

Wheat.—Pot culture experiments have proved that the minimum water requirement of wheat is only 8·5″. Experiments at Shahjehanpur have shown that the number of irrigations required for the crop is dependent on the quantity of rainfall during the later part of summer (September-October) and during winter (December-January). If this is sufficient and well distributed, the crop may not require any extra irrigation. As such conditions are rare generally two irrigations with 60,000 gallons per irrigation (about three acre inches) have given the best yield. Much detailed work has been carried out at the Agricultural Research Stations and the results have definitely shown that the normal and optimum field duty is 200. Higher quantity of water does give increased yields but this increase is not proportional to the extra water added. On sweet lands 12.8 to 14.4 acre inches are sufficient to give normal yields, while Kalar lands require about 18″. It has been further found that provided an adequate soaking dose for sowing purpose is given, the subsequent distribution of irrigation water has not got much effect on final yields.

Sugar cane.—Pot culture experiments have shown that the minimum water requirements of sugarcane is 45″, though varieties of sugarcane differ very widely among themselves in their relative water requirements. As regards the quantity of water per irrigation 80,000 gallons per acre (about 4 acre inches) is supposed to give the best results. The practice of the cultivators to fill the field to the maximum at each irrigation is thus very wasteful. In South India normal irrigation consists of 7 waterings. The irrigation experiments made in the Uttar Pradesh have shown that the frequency of irrigation is more important in its relation to yield than the depth of water at each irrigation. The physiological studies have shown that the critical period in the life of sugarcane crop extends from the middle of May to the end of June with regard to its water requirements. Under conditions obtaining in the Bombay province, it was found that 95 acre inches inclusive of rainfall over a period of 12 months would be the minimum water requirement. It has also been found that the water requirement of the crop is not the same throughout all the stages of the plant life, it being the highest at the vegetative and flowering stages and lowest at the maturity stage. Experiments carried out at Barabanki and Bulandshahr in the Uttar Pradesh show that 5″ depth of irrigation at two weeks interval gave the maximum yield. For Bulandshahr it was significantly borne out that maximum yield was obtained with 4″ depth of irrigation at three weeks interval.

Cotton.—The water requirement for normal varieties of cotton approximates 24″ to 28″. It has been found that the first irrigation should be given 3 to 4 weeks after sowing, as delaying it definitely affects the yield while no irrigations are necessary after 15th of October for normal varieties. For late maturing cottons, however, later irrigation would be a necessity. The Sind physiological investigations which have also been conducted for similar long periods have shown that while there is a longer relationship between the quantities of water and yield, the Barrage duty of 100 is considered to be the optimum as it is just sufficient for crop growth without materially leaving any residual water. The sowing season of the cotton crop extends from April to middle of June and the crop finishes off by the end of December.

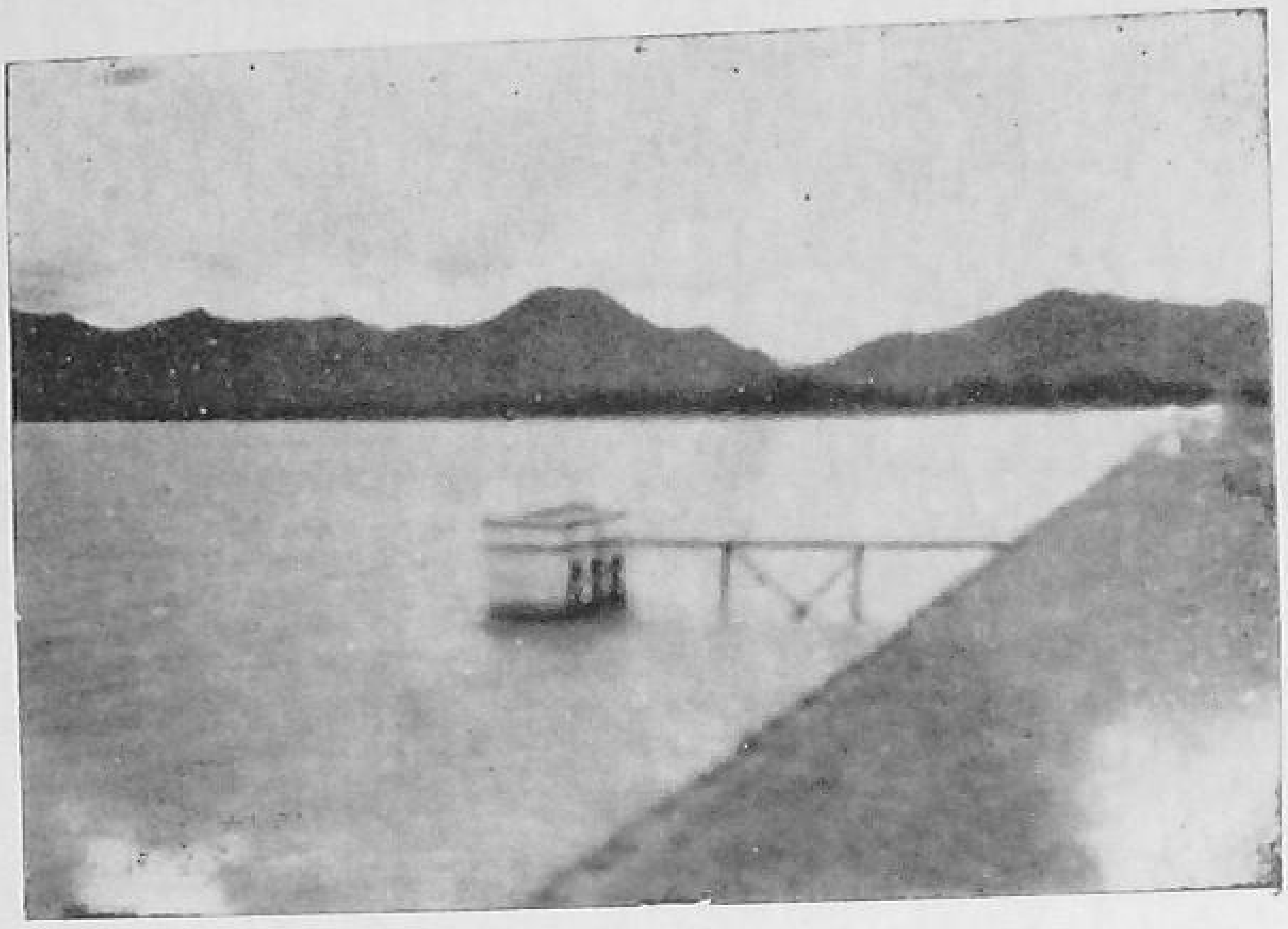
CHAPTER IV

SOURCES OF IRRIGATION WATER

The water cycle is a Nature's well-known phenomenon. Water from the vast seas rises as vapour and saturates the wind with moisture. This moisture-laden wind, in turn, when it reaches the land, drops its load and we have the rainfall. In high altitudes, this moisture gets frozen which gives rise to the snow-peaked mountains. From these mountains, as the snow melts, water finds its way into what are known as "snow-fed rivers" from where it escapes into the sea. The rain water divides itself into several parts. One portion directly flows into small drainages and thence into big rivers which ultimately carry it to the sea. Another portion is absorbed by plants and forests. A third component escapes under-ground and gives rise to what we term as "ground water". The ground water itself gets sub-divided. If the sub-soil is porous with underlying impervious strata, the water collects into under-ground reservoirs. From these reservoirs, this could be picked up directly through percolation wells. There are several under-ground channels, which take off from these ground reservoirs and under favourable geological conditions these channels may cross a valley. Here the ground water appears as a stream flow *i.e.*, the stream continues to have flowing water even after cessation of rains. We call these as perennial streams. Where the under-ground channels do not cross a stream, a regular flow is induced in the sub-soil for long distances. This flow could also be picked up by open wells or tubewells. Both the stream water and the ground water ultimately find their way into the sea and Nature's cycle is maintained.

Water could be tapped at various stages of the cycle for artificial application to crops. The rain water could be directly stored in reservoirs and released for irrigation. Under this category, we have the several major and minor tanks in Southern and Central India, brief description of which appears earlier. The other source of irrigation is the stream or river water, where we get flowing water even after the rains. The canal systems in the Northern India belong to this class of works. Lastly, where all other supplies fail, we have the ground water—sometime Nature's unlimited source of water supply. Wells and tubewells come under this category. As distinct from tank or canal water which flows into the fields by gravity, the ground water has to be lifted by manual or mechanical means which entails a good deal of recurring expenditure. Even so, lifting of

water from wells for irrigation purposes, is an extremely common practice and forms the bulk of minor irrigation works in India. The reason is that certain topographical conditions must be satisfied before a canal or tank can be sited in a particular locality and it is not always possible to get such sites in many places. A well, on the other hand, could be sunk at any place where there is store of ground water. The three different classes of works will now be dealt with in detail in the succeeding chapters.



Russelkunda Irrigation Reservoir, Orissa

Harsi Reserveir in Madhya Barath

CHAPTER V

THE RAIN WATER—STORAGE RESERVOIRS AND SUBMERGING TANKS

The function of an irrigation tank is to store the rain water during rains and issue that water in a regulated manner when natural precipitation fails to meet the water requirements of crops. The essential features of a tank are:—

(i) a tank bund which impounds the rain water. It is generally of earth but sometime masonry dams are also constructed;

(ii) a sluice gate to let out water for irrigation. A canal takes off from the sluice gate and supplies the tank water to the fields;

(iii) waste-weir to dispose of the surplus flood discharge which the tank cannot store.

MAIN FEATURES OF DESIGN

The Catchment Area.—The drainage area which feeds a tank is known as its catchment area. The first requirement of a tank is that it should have ample catchment so that its filling is assured. The actual fall on a catchment is determined from the rainfall *i.e.*, if there is a monsoon rainfall of 40″, the total fall per square mile of catchment would be 93 million cu. ft. of water. All this water does not flow into the reservoir. As stated earlier, a portion escapes underground, some is absorbed by plant and forests, another portion evaporates and the remaining water appears as stream flow which goes to fill the tank. This quantity of water is known as the yield. The relation between yield and total fall on a catchment depends upon several factors. Thus if the rainfall is heavy, the percentage of yield would be higher than it would be for a smaller rainfall. Similarly, if the catchment is porous or sandy, most of the water would disappear underground and very little would be available for filling the tank—in other words the yield would be small. On the other hand, if the catchment is sloping more quantity of water would come into the tank than if it were flat and bunded. The more important factor is the intensity of fall. Thus the yield from a rainfall of 8″ in 6 hours would be much more than from a rainfall of 20″ spread over a period of one month. In actual practice it is difficult to take into consideration all these varying factors for purposes of calculating the yield from a particular catchment. Generally, emperical rules based on actual observations are followed. The most

important of these is the Binni's percentage derived from observations made on the reservoirs of Madhya Pradesh. According to this the yield per square mile of catchment for varying rainfall intensities would be:—

Rainfall	Percentage Yield
0-10	7
10-20	12
20-30	22
30-40	32
40-50	41
50-60	47
60-70	52
70-80	56

For major irrigation tanks with big catchment areas, the wide differences in the physical features of different parts of the catchment tend to neutralize each other. In all these cases, the total rainfall is the main determining factor for the yield and it is generally possible to anticipate this with a fair degree of accuracy by applying Binni's percentage. For smaller catchments feeding minor irrigation tanks proposed in the Community Development Projects, these varying characteristics of individual catchment come into prominence and in following the Binni's table as a general rule, one is apt to make serious mistakes. In such cases, it would be desirable to study the topography of the catchment, the condition of the feeder Nala (*i.e.*, if it has got a big cross-section, it would mean that it brings lot of water from the catchment), the nature of soil etc. etc. Necessary allowances should be made in calculating the yield for these peculiar conditions of the catchment.

When irrigation is required during the monsoon season the tank is usually designed to store only a portion of the above calculated yield. The proportion of the actual storage to the total yield is known as tank percentage. Thus a 50 per cent. tank would, in years of normal rainfall, fill up twice. It is obvious that a low percentage tank, because of the fact that it would fill up several times a year, would be capable of irrigating a larger area than a tank of the same storage but of a higher percentage. The reason is that as irrigation proceeds and the reservoir gets emptied, it is filled up again by the subsequent showers. Each unit of storage would thus make a much larger quantity of water available for irrigation. In consequence, the tank duty per million cu. ft. of storage will increase with the fall in the tank percentage. Tanks for monsoon irrigation should ordinarily be designed for two to three fills (*i.e.*, 30 to 50 per cent.) Figure 1 shows the relationship between the tank duty and the tank percentage as adopted in the State of Madhya Pradesh. In that State the rainfall is 40″ to 50″ and a tank duty of 20 to 30 acres per million cubic ft. of storage is adopted for the monsoon paddy crop.

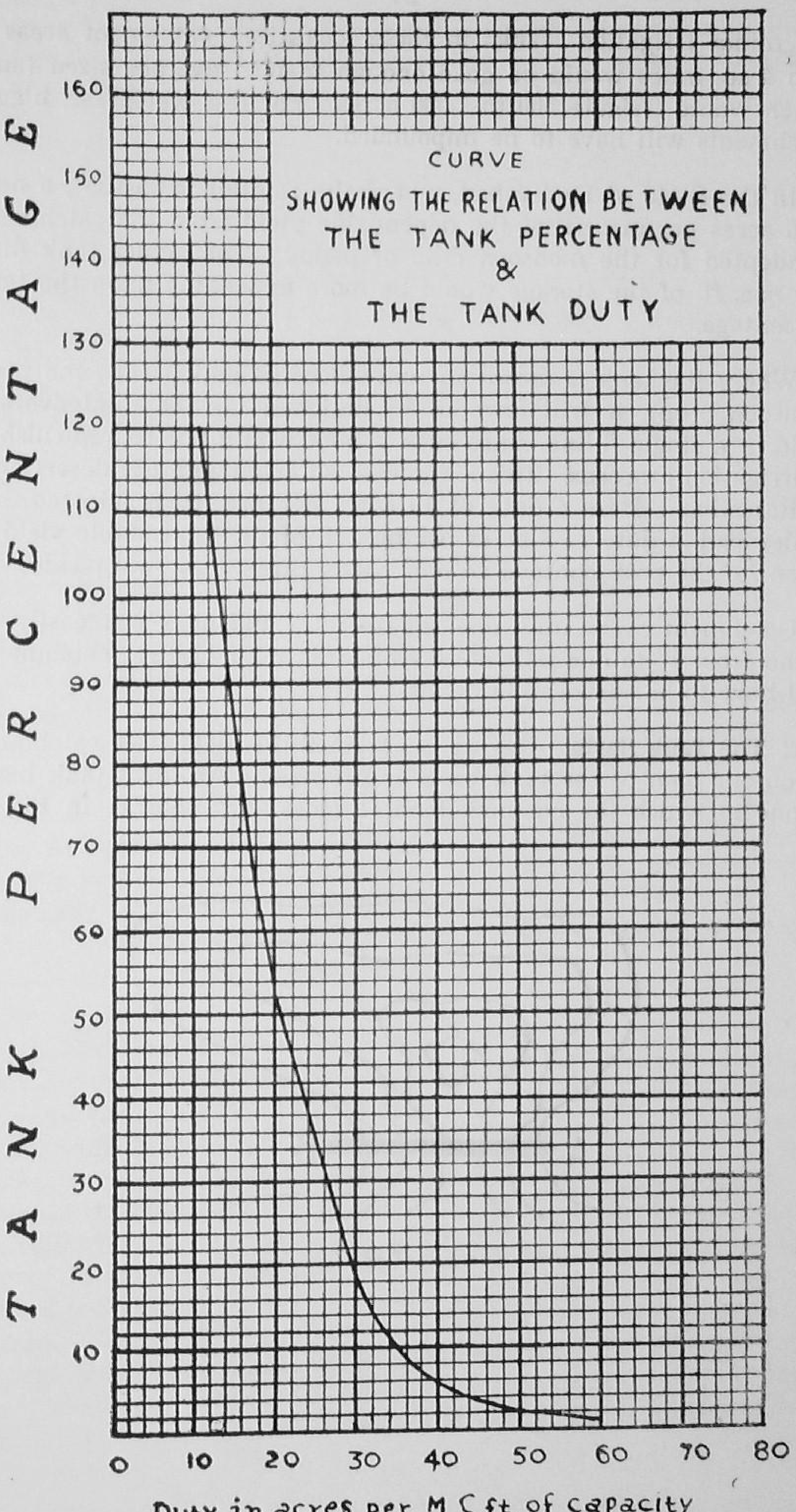

CURVE
SHOWING THE RELATION BETWEEN
THE TANK PERCENTAGE
&
THE TANK DUTY
TANK PERCENTAGE
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
10
20
30
40
50
60
70
80
Duty in acres per M C ft. of capacity

Fig. 1

For minor irrigation tanks in these localities, catchment areas of 1 to 4 sq. miles would be good enough to feed average sized tanks which would irrigate 500 to 1,000 acres. For less wet areas, bigger catchments will have to be impounded.

In the State of Hyderabad, where the rainfall is 30—35", a duty of 6 acres per m.c. ft. of the dependable yield from the catchment, is adopted for the monsoon crop of paddy. The actual tank duty per m.c. ft. of the storage would be more depending upon the tank percentage.

Where the tank is meant for only *Rabi* irrigation (*i.e.*, the post-monsoon crop), it will have to be designed for the total average yield in a single filling, since such a tank would not be replenished as irrigation proceeds. This is the case with tanks in the desert area of Rajasthan. Here a duty of 10 acres per m.c. ft. is adopted. In Hyderabad, a duty of 3 acres per m.c. ft. of the dependable yield is taken for the post-monsoon crop of paddy (known as *Tabi* paddy).

Catchment areas upto one square mile and over can easily be found from 1" to one mile survey of India map. Lower catchments will have to be surveyed in detail.

2. **The Tank Basin.**—Having ascertained the yield from catchment a contour plan of the tank basin is prepared. An ideal tank basin is one in which the contours tend to close as shown in Fig. 2.

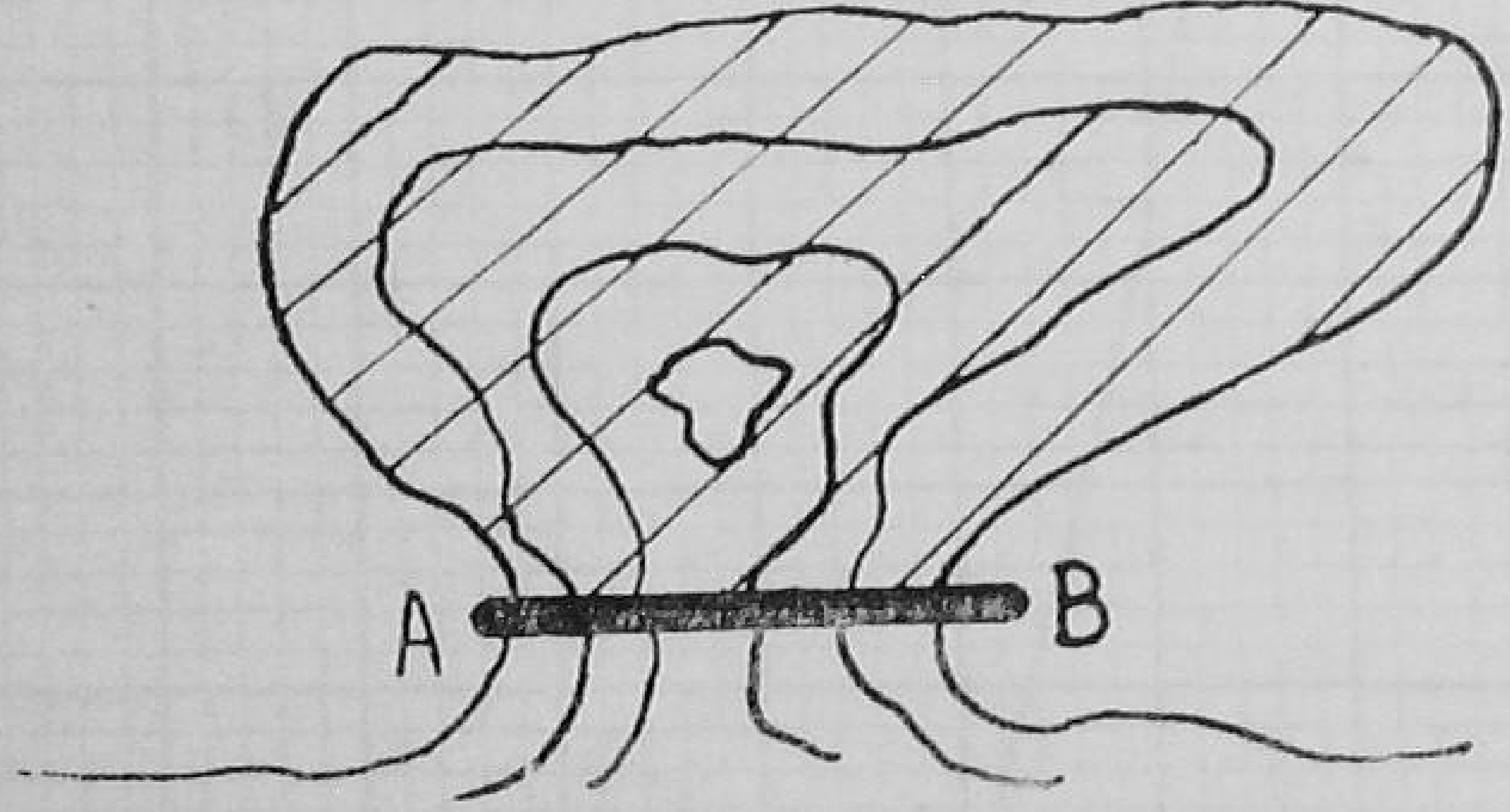


Fig. 2

Thus by constructing a short length of bund along A.B., the entire storage (shown shaded) could be impounded. If the tank basin is as shown in Fig. 3.

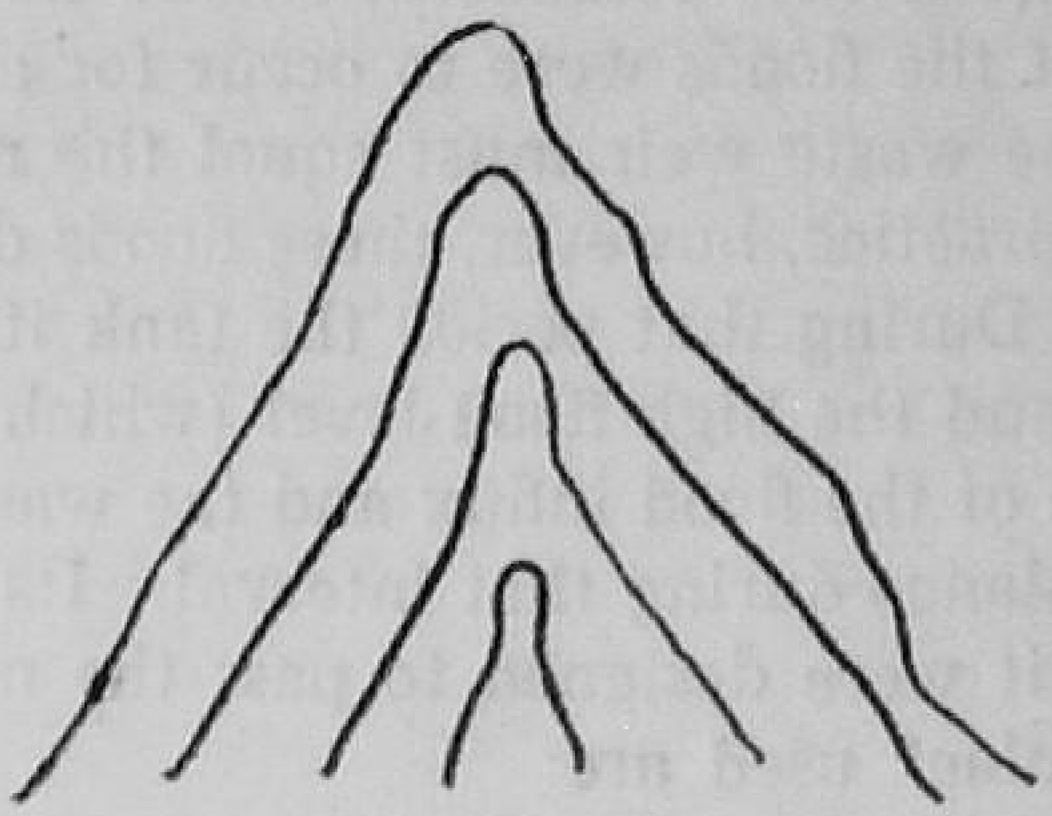


Fig. 3

a much longer bund and also of greater height will be needed to construct a tank of the same capacity. Not only should the contours close but they should be spaced as far apart as possible——in other words a flat tank basin will have a much bigger storage than one with a steeply sloping basin, for the same size of the tank embankment. The contour plan of the tank basin can easily be prepared by running a few lines of level in the basin. From this plan capacity of the tank at any level can be determined. The limiting factor for the irrigating capacity of the tank is either the catchment area or the commanded area. If the catchment area is not large, we cannot have too big a tank. Similarly, if the commanded area proposed for irrigation is small, it would be no use constructing a big tank.

3. **The Waste Weir.**—As has been stated above, a reservoir is not usually designed to store the entire run-off from its catchment area. The surplus water coming into the tank after it is full is allowed to escape through the waste weir which consists of a simple masonry bar, generally flush with the ground level but sometimes combined with a drop wall. Water passing over the waste weir is led through a spill channel and eventually drops into the main Nala. Since the crest of the waste weir is at the full tank level which is much higher than the Nala bed level, the spill water has to pass over a big drop. The waste weir has, therefore, to be located preferably in rocky soil or hard ground to prevent scour. Should hard rock be not available, suitable fall or falls have to be provided. If there is a natural saddle suitably located, it would be an ideal site for the waste weir.

Major Garrets formulae are commonly used for the design of waste weir for all major irrigation tanks and the same could be adopted for the design of waste weir for minor irrigation tanks also. These calculations take into consideration the flood storage capacity of the reservoir. If the floods were to occur for an indefinite period, the discharge of the waste weir must equal the maximum flood discharge. In actual practice, however, these floods do not last for more than a few hours. During that period the tank itself stores between its full tank level and the high flood level (which is its flood storage capacity) a portion of the flood influx and the waste weir is required to pass only the balance during that interval. Its length would thus be shorter than if it were designed to pass the maximum flood discharge. The formulae* used are:

$$B = \frac{Ti}{2hA} \quad \text{and where} \quad l = \frac{ni}{ch^{\frac{3}{2}}}$$

T=Duration of flood in seconds.
i=Maximum flood influx.
h=Flood lift in feet.
A=Mean water spread area in square feet between the High Flood Level (H.F.L.) and Full Tank Level (F.T.L.)
l=length of weir required in ft.
c=co-efficient depending on the form of weir.

B and n are connected by the following table.

B	n	B	n	B	n	B	n
.50	0	.74	.655	.98	.860	1.22	.938
.52	.095	.76	.682	1.00	.870	1.24	.942
.54	.178	.78	.706	1.02	.879	1.26	.946
.56	.252	.80	.728	1.04	.887	1.28	.950
.58	.318	.82	.748	1.06	.894	1.30	.954
.60	.377	.84	.766	1.08	.901	1.35	.960
.62	.429	.86	.738	1.10	.908	1.40	.965
64	.477	.88	.799	1.12	.914	1.50	.974
66	.520	.90	.813	1.14	.920	1.60	.981
.68	.559	.92	.826	1.16	.925	1.80	.990
.70	.594	.94	.839	1.18	.930		
.72	.626	.96	.850	1.20	.939	Infinate	1.000

*Pages 326 and 327 of Buckley's Irrigation Pocket Book—1928.

If the flood duration be assumed as three hours (for minor catchments, this would be a safe period) the length of waste weir in feet for various flood lifts, water spread area and catchment area, would be as follows:—

Flood lift=1 ft.

Mean water spread Area in acres .	40	60	80	100	120	140	160	180	200
Catchment are in sq. miles									
·5	180	130	149	116					
1·0	308	308	301	290	272	247	215	176	
1·5		417	417	413	403	389	372	348	

Flood lift=2 ft.

Mean water spread Area in acres .	40	60	80	100	120	140	160	180	200
Catchment area in sq. miles									
1.0	107	97	77	47	10.5				
1.5	148	143	132	113	88	56	14		
2.0	184	182	175	163	144	120	91	56	

In the above calculations the co-efficient c has been taken as 3·25 which holds good for a sharp-crested weir with a free overfall. For minor irrigation tanks generally, a flush masonry bar is provided as a waste weir. Here the weir is neither sharp-crested nor has a free overfall and during high floods the configuration of the water levels would be somewhat as shown below:

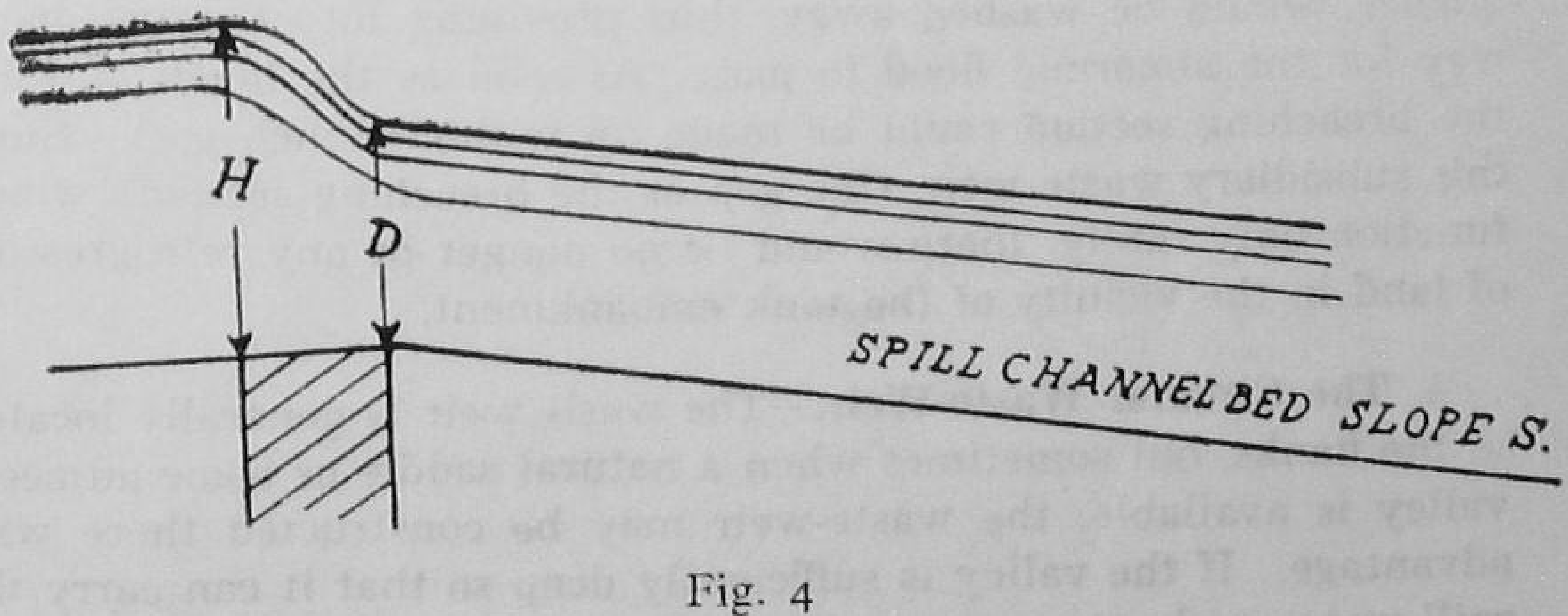


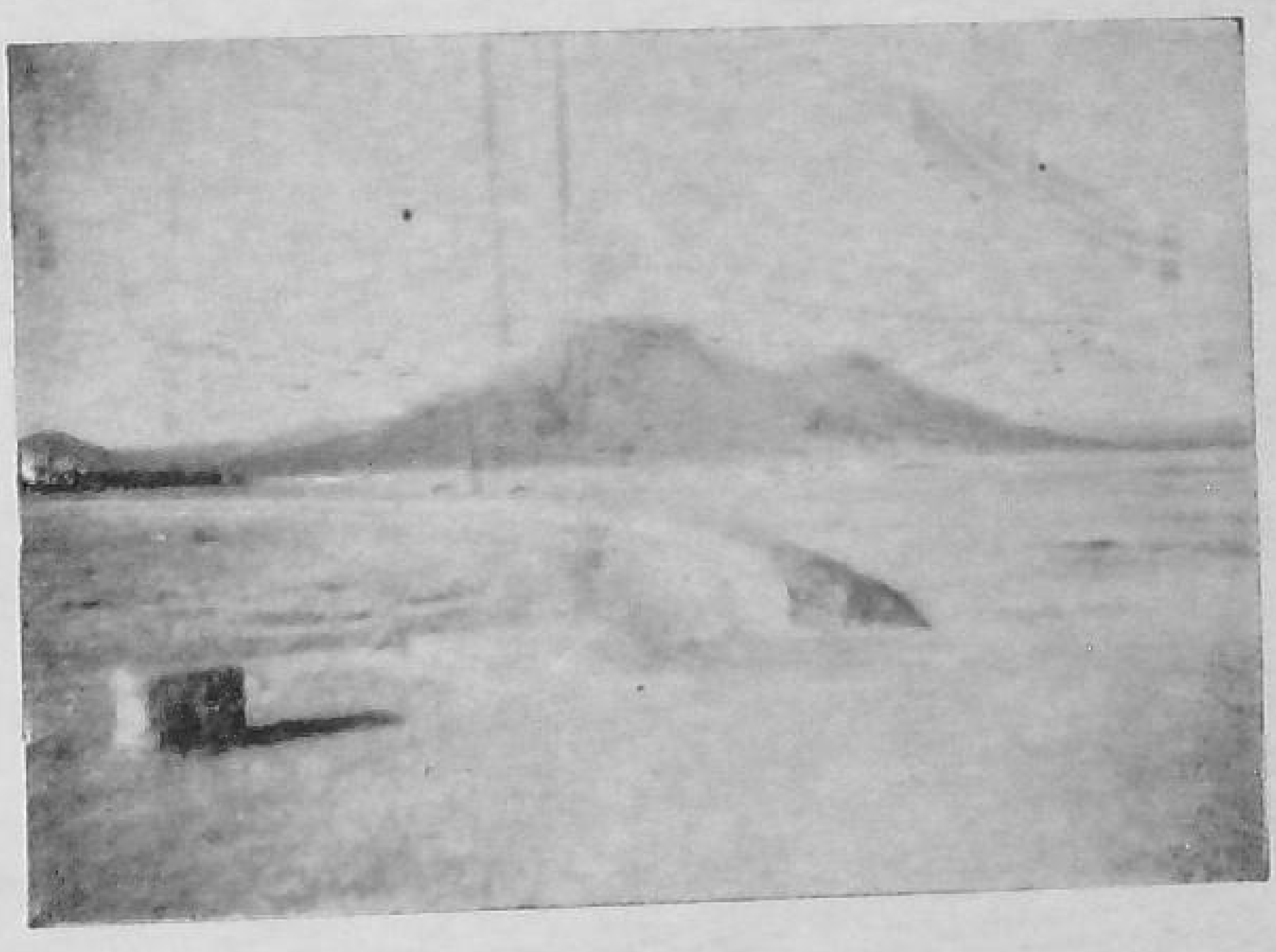
Fig. 4

For a given slope in the bed of the spill channel (which would be governed by the nature of soil through which it passes), the depth of water D could be determined. From H (which is the flood lift) and D, the extent of submergence could be calculated. Generally, the submergence would not be more than ·6 and in that case by assuming the value of 2·5 for the co-efficient c, the flood discharge could be calculated with a fair degree of accuracy e.g. the length shown in the above table will have to be multiplied by the factor

$\frac{\text{"}3.25\text{"}}{2.5}$.

The waste weirs of all major irrigation tanks are designed for maximum flood conditions, as the slightest risk would endanger the safety of its dam. It seems, however, unnecessary to adopt the same safety measures for minor irrigation tanks and their waste weirs could possibly be constructed of much shorter lengths. Maximum flood conditions as derived from Dicken's formulae occur once in several years. While for major irrigation works, even these rare occurrences have to be guarded against, as the risk involved is very great, for minor tanks such elaborate waste weirs designed for maximum flood conditions do not appear to be necessary. When suitable sites are not available the cost of the waste weir is sometimes quite out of proportion to the entire cost of a minor irrigation tank project. The best way, it seems, would be to design the regular waste weir for minor tanks for half the flood discharge derived by the Dicken's formulae, and a gap left at one extremity of the bund. The ground level here should be about a foot higher than the crest of the main waste weir. The latter will dispose of all normal floods successfully. For abnormal floods, extra water will escape through the gap without causing any appreciable damage. Alternatively, a breaching section 1 or 2 ft. high could be provided in the gap portion. When the floods exceed the normal flood for which the waste weir has been designed, it would first encroach upon the free-board which should not matter much for a minor tank. If the floods increase still further, water would over-top the breaching section which, in consequence, would be washed away, thus providing lot of extra water way for the abnormal flood to pass. As soon as the floods subside, the breaching section could be made up without much cost. Since this subsidiary waste weir (the gap or the breaching section) would function only rarely, there would be no danger of any retrogression of land in the vicinity of the tank embankment.

4. **The Circular Waste-Weir.**—The waste-weir is generally located in the flanks, but sometimes when a natural saddle or some adjacent valley is available, the waste-weir may be constructed there with advantage. If the valley is sufficiently deep so that it can carry the spill water without causing submersion of the waste-weir, the cost of



The Circular Waste-weir in the Harsi Reservoir, Madhya Bharat.

the surplusing arrangement could be considerably cut down by constructing a circular waste-weir. By this arrangement the waste-weir could be made of the required length without the need of enlarging the spill channel. Such waste-weir could be occasionally seen on the upstream of a road culvert where the road itself forms the tank embankment.

Latest experiments have, however, shown that the full length in a circular waste-weir is not as effective as in a straight waste-weir because the co-efficient of discharge in the former case decreases to some extent. The photograph on the previous page shows the circular waste-weir in the Harsi reservoir, Madhya Bharat.

5. **Losses in storage due to evaporation and absorption.**—The evaporation and absorption losses differ from place to place and also from season to season. The losses in the rainy season are also partly due to leakage as the reservoir is full and it operates under high heads. For determining the effective capacity of the reservoirs, losses may be calculated at the following rates:—

Cold Weather—	3″ per month	=12″ for 4 months.
Hot Weather—	10″ —do—	=40″ for 4 months.
Monsoon —	5″ —do—	=20″ for 4° months.
		Total =72″ *i.e.* 6 feet

6. **Silting of Reservoirs.**—Records available from some of the tanks in the Bombay Presidency, indicate that the loss in storage due to silting of reservoirs is of the order of 15 to 25 per cent. during the period of 30 to 40 years. An allowance of 10 per cent. as dead storage, has been recommended by Strange for fixing the sill level of reservoirs.

EARTHEN DAMS FOR STORAGE RESERVOIR*

In choosing the site of a dam, it should be remembered that the cost of reservoir project is determined mainly by the longitudinal section of the site of the embankment. The cost of a dam varies roughly as square of its height. Dams which are long but low, are frequently cheaper, and are safer than shorter and higher ones.

The Dam Foundation.—The essential conditions for the foundation of an earthen dam are that the soil should be compact, that it will not yield when wet, and that it will not settle nor slip under the weight of the dam. It is most important to drain the ground itself at the rear of the down stream toe of a high earthen dam. It is highly desirable to prevent the sub-soil at the rear of the embankment from becoming sodden, for, if it does, the damp ground will tend to give way under the weight of the dam. It is a good plan,

* Extracted from Buckley's Irrigation Pocket Book—1928—pages 558—563

therefore, to slope off the ground just below the outer toe of the embankment, for a width of about 30 ft., at an inclination of about 1 in 10, and to cut at the foot of this slope a small "surface drain", designed to carry away at once any rainfall flowing off the slope of the dam. A short distance away from the "surface drain", a "down-stream drain" should be constructed parallel to the embankment (See fig. 5). The object of this "down-stream drain" is effectually to drain the sub-soil both on the down-stream side of the toe of the dam and for some distance below the dam itself; it also serves to carry off the water from the "foundation drains" immediately under the embankment.

The maximum amount of percolation through the embankment will probably occur along the base of the dam at its junction with its foundation. It is necessary to take great precautions at this point of the work, first, to intercept as far as possible, any flow there may be and secondly as the flow cannot be entirely prevented, to drain off without injury to the dam, the water which has to be passed away. Fig. 5 shows the plan for doing this. On the up-stream side of the foundation of the dam is a series of small puddle trenches, parallel to the main central one, and on the down-stream side is a similar series of trenches or "foundation drains". Those on the up-stream side are filled with as retentive material as can be obtained, but those on the down-stream side with porous material. The "foundation drains" are designed to draw off uniformly and thoroughly the water from the whole of the overlying part of the dam. These drains should be cut in disconnected lengths of about 300 ft. each, falling uniformly to their lower ends. There they should run into cross drains of similar section, cut at right angles to the axis of the embankment. These cross drains should be carried in covered slab vents, well below the "surface drain" into the "down-stream drain" which forms their outfall.

The whole seat of the dam, on each side of the puddle trench should be stripped of unreliable soil and excavated into a series of large furrows parallel to the axis of the dam. These furrows should have their troughs immediately above the "foundation trenches and drains". The benches cause the dam to rest on a series of planes slightly inclined towards the centre of the embankment which will tend to make the layers of earth-work settle towards the axis of the dam. This will increase the stability of the work.

(2) **The Puddle Trench.**—The object of a puddle trench is to interpose a water-tight "septum" which will prevent the passage of water and will keep all the material down-stream of it as dry as possible. The position almost invariably chosen for the puddle trench is on the centre line of the embankment. The depth to which

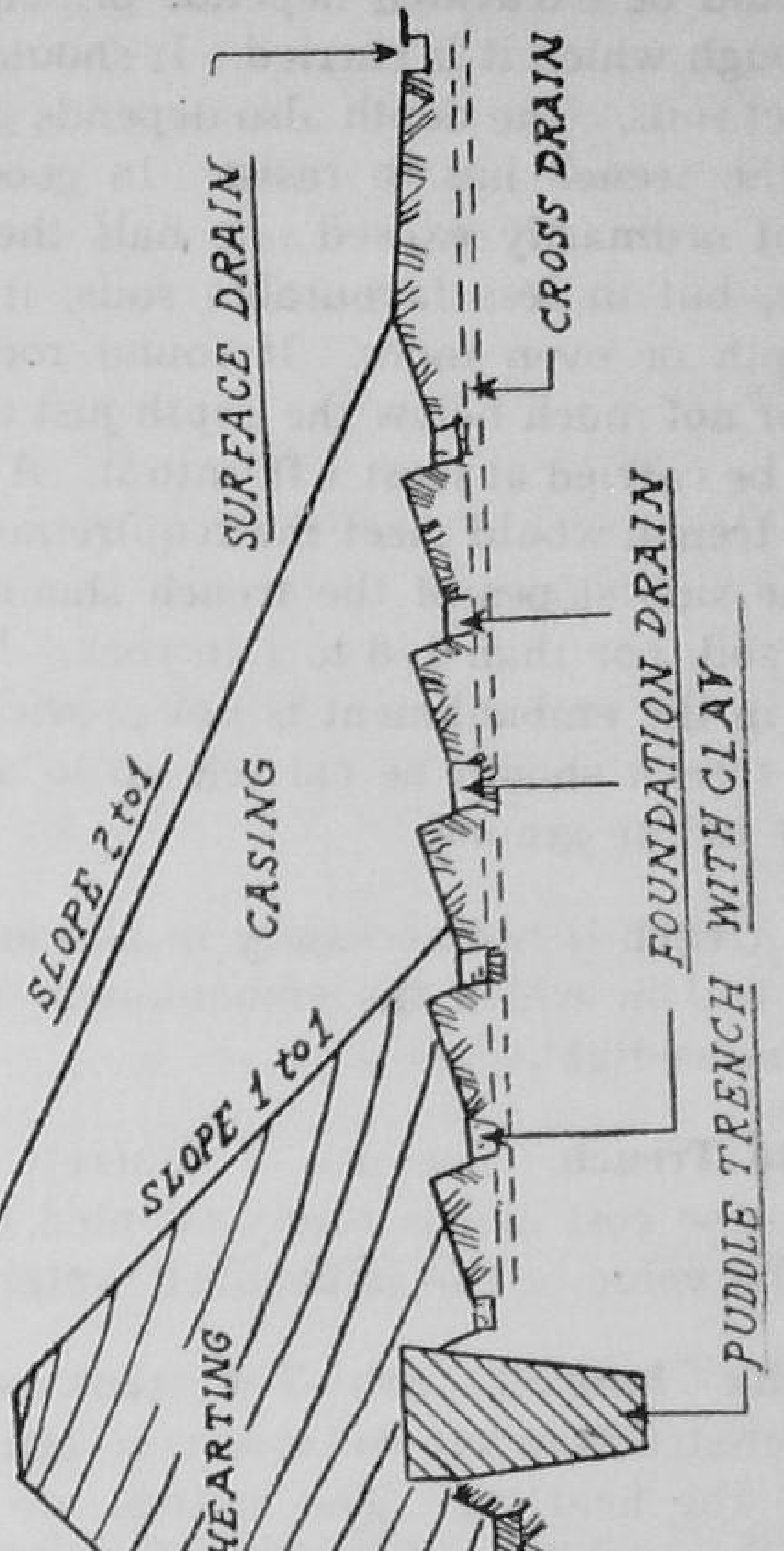

FULL TANK LEVEL
SLOPE 2 to 1
CASING
SLOPE 1 to 1
HEARTING
SLOPE 1 to 1
SLOPE 2 to 1
CASING
SURFACE DRAIN
CROSS DRAIN
FOUNDATION DRAIN
FOUNDATION TRENCHES
WITH CLAY
PUDDLE TRENCH WITH CLAY

Fig. 5

the trench should be excavated depends principally on the porosity of the soil through which it is carried. It should be deeper in porous than in compact soils. The depth also depends greatly on the head of water which the trench has to resist. In good compact soils the depth need not ordinarily exceed one half the depth of the full supply storage, but in less favourable soils, it may be carried to twice that depth or even more. If sound rock is found near the ground level, or not much below the depth just mentioned, the trench should always be carried at least 1 ft. into it. A bottom width of 6 ft. for the puddle trench would meet the requirements of a minor irrigation tank. The side slopes of the trench should not be cut steeper than $\frac{1}{4}$ to 1 in soil, nor than 1/8 to 1 in rock. In those cases, where a puddle wall in the embankment is not provided, the height of the puddle in the trench should be carried up to about 2 ft. above the natural surface of the ground.

The puddle trench is not necessary in all cases. It may be omitted where the soil in which the embankment is to be constructed, is reasonably water-tight.

(3) **Concrete Trench.**—The use of concrete, instead of puddle adds greatly to the cost and is rarely adopted in reservoir embankments unless the value of the impounded water is very great.

(4) **The Tank Embankment.**—The tank embankment which forms the super-structure of the reservoir consists of two parts—the casing and the hearting. The casings are designed to protect the interior of the dam from external influences. On the up-stream side the casing is necessary to form a firm and insoluble foundation for the pitching. On the down-stream side, the casing should be a covering which will not crack by the sun nor gutter by rainfall. The material of the casing should be shaly free from clay as far as possible. The hearting consists of clayey material and is meant to intercept the percolation of water even against high heads. For minor tanks the top width of the dam could be kept from 5 ft. to 10 ft. depending upon the height. Both the up-stream and down-stream slopes of the embankment could be kept as 2 to 1. If the embankment is very high the up-stream slope could be made flatter *i.e.*, $2\frac{1}{2}$ to 1 to 3 to 1. The free board may be kept as 3 feet where the depth of water is 8 feet or less and may be 4 to 5 ft. as the depth increases.

It is the general practice in England, but not in India to have a puddle wall along the central line of the dam; it is placed vertically over, and made one with the puddle trench so as to form with it a water-tight septum extending throughout the dam from below bed rock level to above high flood level. The object of it is to in-

tercept completely any infiltration which may have penetrated so far. The thickness of the base of the puddle wall is usually kept one-third of its height while the top width is made one-half to two-thirds of its base. The puddle wall is economical in so far as it limits the amount of the material to be made water tight, but its use implies that the earth work upstream is not reliable. It is most undesirable to have so large a mass of the dam saturated with water.

The American engineers are greatly in favour of masonry core walls for earth dam. The relative advantages of puddle and masonry core walls have been much disputed. On the whole, it appears that masonry is superior to puddle but more expensive.

(5) **Pitching.**—In a large reservoir, the action of the wind will cause waves, which if they are allowed to break on the unprotected embankment will very soon wash away the earth work. The object of the pitching is to protect that surface. The pitching stones are firmly bedded on a layer of sand, hard moorum or quarry spauls, at least, 6″ in thickness and are laid with their broadest ends downwards. If the direction of the tank embankment is favourable with respect to the wind direction, no pitching may be needed.

REPAIRS AND REMODELLING OF OLD EMBANKMENTS*

Junction of earth work requires very careful construction. In actual practice, generally, the following cases will be met with:—

(i) **Cross-Sectional Junction.**—These should be made as shown in Fig. 6. All the loose surface earth should be entirely removed and as shown in plan, the junction should be made with slip tongues

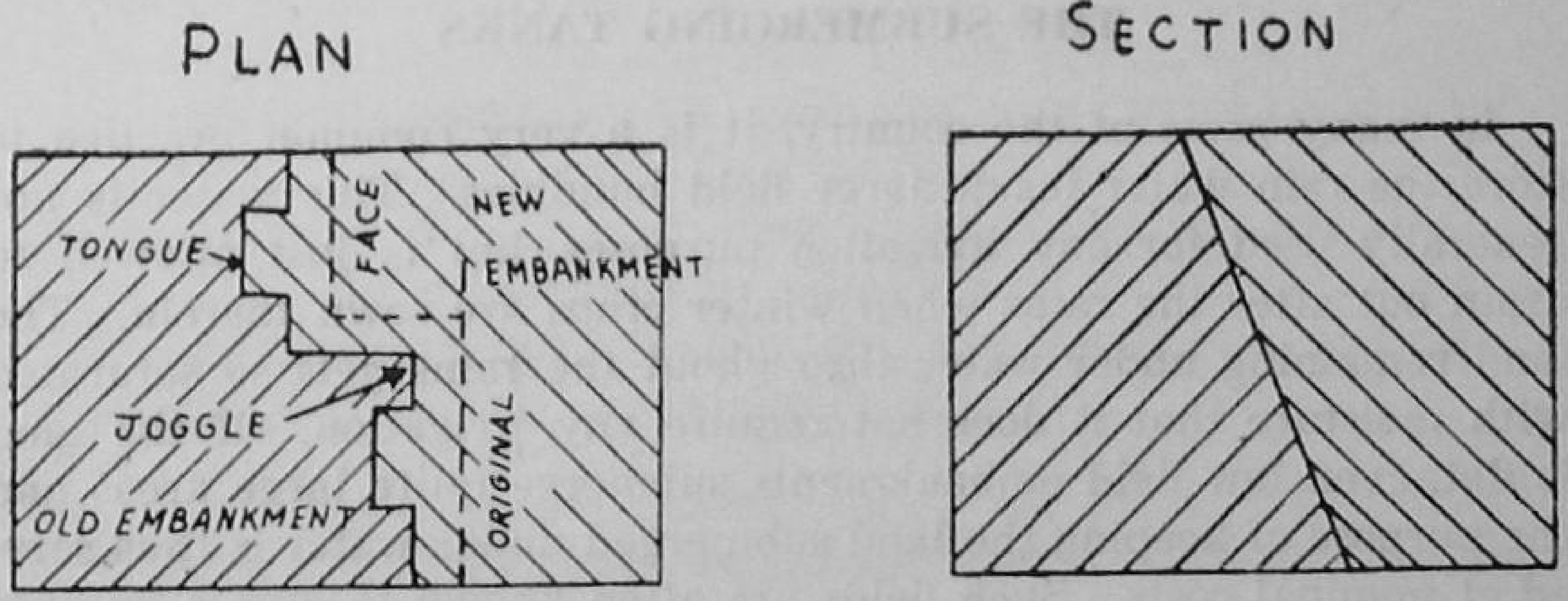


Fig. 6

*Indian storage reservoirs with earthen dams by W.L. Strange .

and joggles inclined up the face of the slope to allow the new embankment to settle tightly on the old one.

(ii) **Longitudinal Junction.**—These junctions should be as shown in Fig. 7 by benchings of irregular size. This is designed so as to prevent the formation of slipping plane.

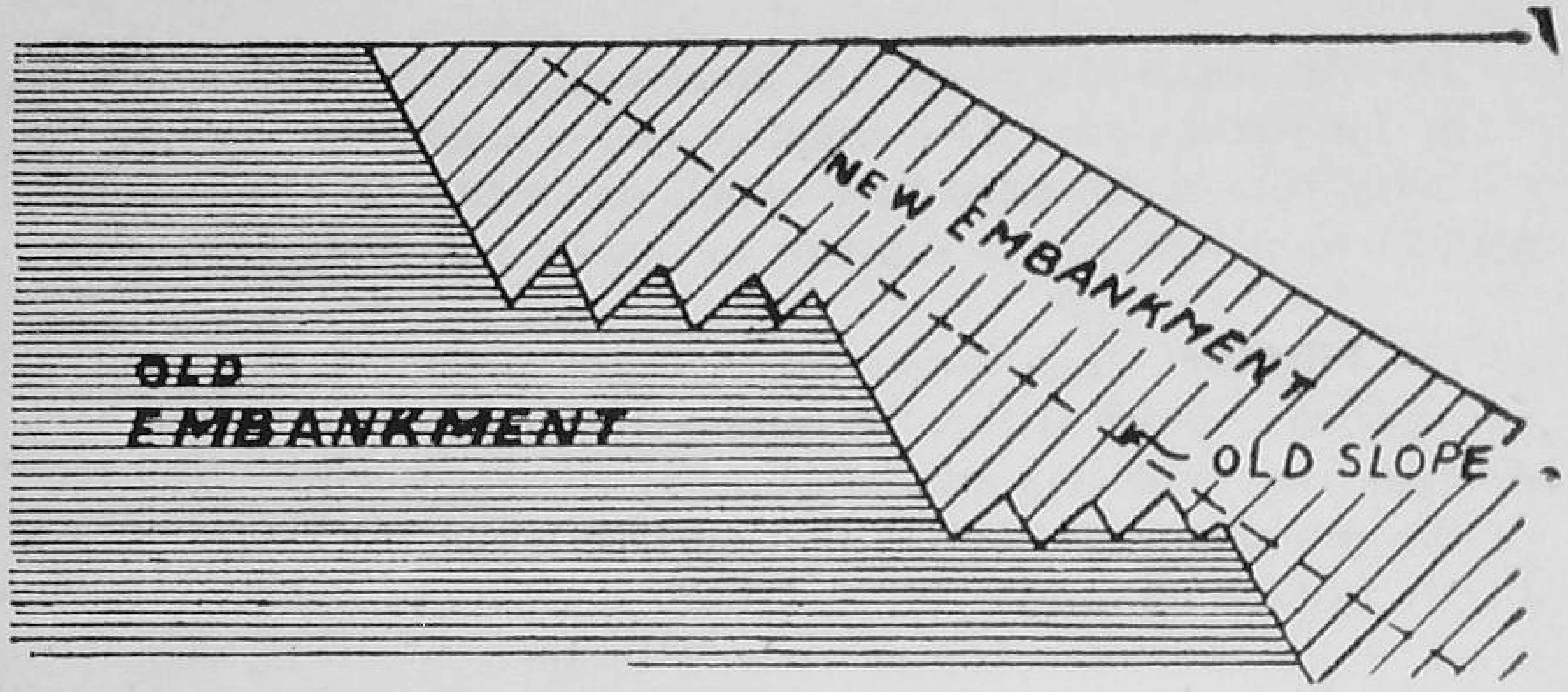


Fig. 7

(iii) **Additions to height.**—Whenever new earth work has to be added to the top of the old earth work *e.g.* when an old dam with a large top width has to be raised, in addition to removing the old surface, one or more key trenches say 4 ft. wide at the base, by 3 ft. deep should be excavated in the old work with slightly sloping sides and parallel to its centre line. These should be filled with the most retentive material thoroughly rammed before the main part of the earth work is commenced.

THE SUBMERGING TANKS

In many parts of the country, it is a very common practice to store the rain water inside large field bundhies. This water is not generally used for any irrigation purposes, but is just allowed to drain out after the rains when winter crops are sown therein. The land remaining under water throughout the rains gets so saturated with moisture that it does not require any irrigation. If the land is flat, even low field embankments submerge quite large areas and the purpose of keeping the land submerged under water is thus served at nominal costs. Such fields are often known to give a bumper crop of unirrigated wheat. It is a common knowledge that as the water level in an irrigation tank falls, the tank bed is released for

cultivation purposes. No irrigation is done in these tank beds and the yield in many cases is known to be much more than that of the irrigated crop. Such field embankments are a very common sight in the northern districts of Madhya Pradesh (Jabalpur and Sagar). Here the soil is black, which is very retentive of moisture. After remaining submerged under water during the whole of rainy season, this soil retains so much water that there is always an excellent harvest of wheat crop even when there are no winter showers. In Kotah and Bundi districts of Rajasthan also, many such embankments exist—some of these are maintained and managed by Government, but mostly these are owned by the cultivators themselves. In this area, regular escape-weirs have also been constructed on one side of the embankment for the disposal of the surplus flood water. The "Ahars" in Bihar which store water for irrigating paddy fields also function somewhat in a similar manner. Water is let out in October for irrigating the rice fields and the drained out fields in the bed of the Ahars are cultivated with rabi crop. In all these works, it is to be observed that the total quantity of water stored in the tanks has got no bearing on the area benefited. The main consideration is the total area getting submerged during the rains. A flat basin, therefore, will be most favourable for the construction of such an embankment (see also "Flood Irrigation" in Chapter VI).

GRAVITY DAMS* (MASONRY OR CONCRETE)

For minor irrigation tanks, construction of masonry or concrete dams would be rarely found necessary because the earthen dams can be safely constructed upto height of 90 to 100 ft. It is only when the site has a natural gorge with steep side slopes and high flanks with good rocky foundations, that the construction of a gravity dam would be particularly suitable. For the earthen dams, it is sometimes difficult to find suitable site for the waste-weirs while in masonry dams, the flood could be discharged directly over a lower part of the main dam itself. In hilly countries, stones are generally available in plenty while earth work suitable for dam construction is not easily available. In such cases also, conditions would generally favour the construction of a gravity dam. The additional advantage of the masonry dam especially in areas where tanks have a tendency to silt up quickly, is that by providing under-sluices in the lower-reach, the silting could be considerably diminished.

(i) **Foundation and Siting.**—The most important condition for a masonry dam is that it should have sound foundations. Presence of

*Irrigation Engineering by K. R. Sharma.

faults or absence of rocks of adequate strength have to be carefully avoided. It is very necessary to obtain detailed geological data of the sub-soil formation by carrying out exploratory drillings. Many sites could be rejected outright if suitable rock is not available for foundations within a reasonable depth. If indications available from the exploratory drillings are favourable, the best way to site the dam, would be to locate its centre line in such a way that the trace of the toe of the finished dam coincides with the line joining the ridges of the gorge.

(ii) **Profile of low Dam***.—The various causes that lead to the failure of a masonry dam are—

(a) by over turning;

(b) by settlement or crushing of the foundations;

(c) by sheering or sliding on any horizontal plane;

(d) by crushing of the dam material; and

(e) by the development of tensile stress in any part of the dam.

The dam should be designed in such a manner that it is safe against all the above causes when the reservoir is both empty and full. Generally, the dam is strong enough to resist failures against (a) or (c) and in designing a dam, precautions have to be taken to guard against its failure due to the remaining three causes.

Let b be the length of the base of the masonry dam, p the specific gravity of the material (with respect to water), P, total water pressure and W, weight of the dam. R represents the line of resultant thrust which cuts the base at the downstream extremity of the middle third. (Whenever the resultant falls outside the middle third of a horizontal section, the structure will develop tensile stress. As such, this would be the limiting position for the resultant R). When the reservoir is empty, P is zero and W, being the weight of the masonry dam is the only force acting on it. In this condition,

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*Irrigation Pocket Book by Buckley 1948-49—p. 568 and Irrigation Engineering by K. R. Sharma—p. 78.

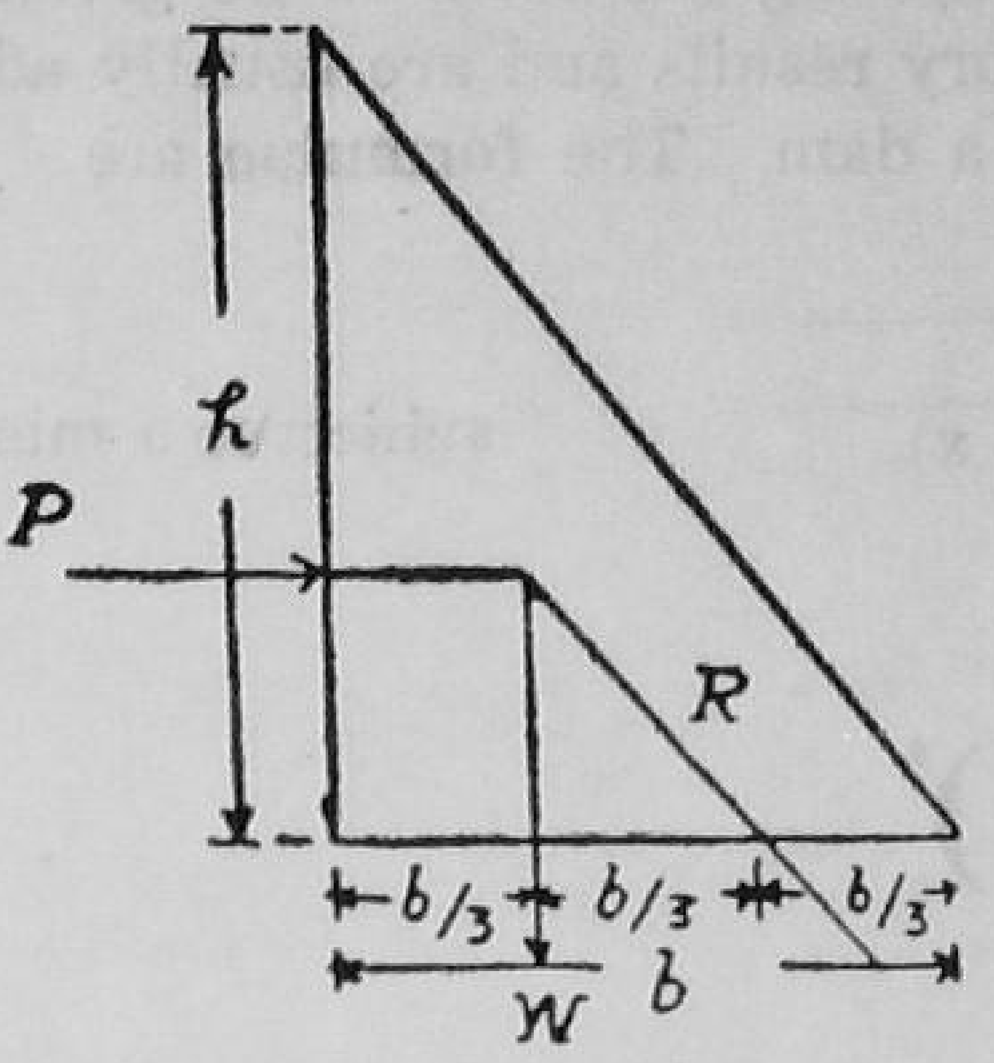


Fig 8

for W to pass through the up-stream extremity of the middle third, the profile of the dam would obviously be a right angled triangle as shown in the sketch. From the triangle of forces

$$\frac{P}{W} \quad \frac{\frac{b}{3}}{\frac{h}{3}} = \frac{b}{h}$$

or $$\frac{b}{h} = \frac{\frac{1}{2}\,wh^2}{\frac{1}{2}\,whb\rho}$$

or $$b^2\rho = h^2$$

or $$b = \frac{h}{\sqrt{\rho}}$$

The value of is p usually assumed as 2:35. The above is only a theoretical profile. In actual practice, it is impossible to adhere rigidly to such a profile as a dam must have a certain crest width and also some free board above the full tank level. The adjustment required to rectify this is to give a slight projection outside the elementary profile, to the lower part of the upstream face. Molesworth's

formulae* for determining the form to be given to a masonry dam give fairly satisfactory results and are usually adopted for fixing the tentative profile of a dam. The formulae are

$$y = \sqrt{\frac{.05x^3}{\lambda + (.03\ x)}} \quad \text{subject to a minimum of } .6x$$

$$z = \left(\frac{.09\ x}{\lambda}\right)^4$$

$$b = .4\ a$$

a = y at a depth of ¼ H.

where H = Height of dam in ft.

x = The depth in ft. of any imaginary horizontal plane from the surface of water.

A vertical being dropped from the top of the up-stream face of the dam; then

y = The ordinate in ft. from that vertical line to the outer or down stream face of the dam, at any depth x.

z = The ordinate in ft. from that vertical line to the inner or up-stream face.

and b = width of the dam at top in ft.

λ = Limit of pressure in tons per sq. ft.

(iii) **Testing the stability of a masonry dam†.**—The easiest method to test the stability of a dam, whose tentative profile has been decided upon by Molesworth's formulae is by graphical method. This is described below:—

The dam profile DEE″ FBAC is drawn to a suitable scale as shown in the drawing as in Plate I. From A, AA′ is drawn at right angles to AC and AA′ is made equal to the water pressure at A which would be proportional to H. A′J is drawn parallel to CA to meet BA produced in J. AC is produced to meet the water line at K. JK and A′K are joined. CM and CC′ are drawn parallel respectively to AJ and AA′ to meet JK and A′K in M and C′ respectively. From CM, CS is cut off equal to the water pressure at C (*i.e.*, in this particular

*Buckley's Irrigation Pocket Book, 1928—p. 569.

†Irrigation Engineering by K. R. Sharma—pp. 88-90.

The Practical Design of Irrigation Works by W.G. Bligh—p. 53.

case it would be $\frac{90}{150}$ x AA') and SD is joined. The area of the triangle DSC gives the water pressure on DC and the area of the trapezoid C'CAA' gives the water pressure on AC. Area of the trapezoid C'CAA'.

$$= \frac{AC}{2}(CC'+AA') = \frac{AC \cos \alpha}{2} \times \frac{(CC'+AA')}{\cos}$$

$$= \frac{AZ}{2} \times \frac{CM + AJ}{2} = \text{the trapezoid AJCM.}$$

Thus the water pressure on the face AC is represented by the trapezoid AJCM also. The whole depth of the dam (H) is divided into 5 laminae of equal depth by horizontal base lines H/5=30 ft. apart. The weights, centres of pressure and stress as base lines are to be delineated by graphic statics. The method for determining the centre of gravity of a trapezoid is also indicated on the drawing in the case of trapezoid JAWV. The diagonals are joined inter-secting in the point R. JQ is made equal to WR. The centre of gravity of the triangle AQV is also the centre of gravity of the trapezoid JAWV.

From what has been stated above, it will be clear that the pressures P_1, P_2, P_3, P_4, P_5, on the up stream faces of the laminea are proportionate to the area of the horizontal trapezoid opposite to them (shown shaded for the face CW) and will act at right angles to the face of the dam through the centres of pressure, which are the points of intersection of horizontals through the centres of gravity of the pressure areas and the face of the dam. Similarly, the weight W_1, W_2, W_3, W_4, W_5, of each lamina represented by its area would act vertically through their centres of gravity in the vertical lines G_1, G_2, G_3, G_4, G_5. A diagram of forces is then constructed by drawing a vertical line from any convenient point along which ab_1, b_1c_1, c_1d_1, e_1,e_1f_1, representing the weights of the successive laminae are set off. Again from the 'a' in a horizontal direction, a b_2, b_2c_2, c_2d_2, ar set off equal to P_1, P_2, P_3. From d_2 a line d_2 f_2 is drawn parallel to and in the direction of P_4, and P_5. R_1, R_2, R_3, R_4 and R_5 then drawn in the force diagram which represent the resultant pressures of the masonry weights and water pressures of each lamina.

The four dotted cross lines b_1c_2, c_1d_2, d_1e_2, are then drawn joining the lower end of one resultant with the upper end of the next.

A convenient point O is chosen as pole and the diverging rays from O to each of the divisions of the vertical line are drawn,

The verticals through ,G_2,G_3,G_4 and G_5 are produced and from a convenient point "q" in the vertical through G_1

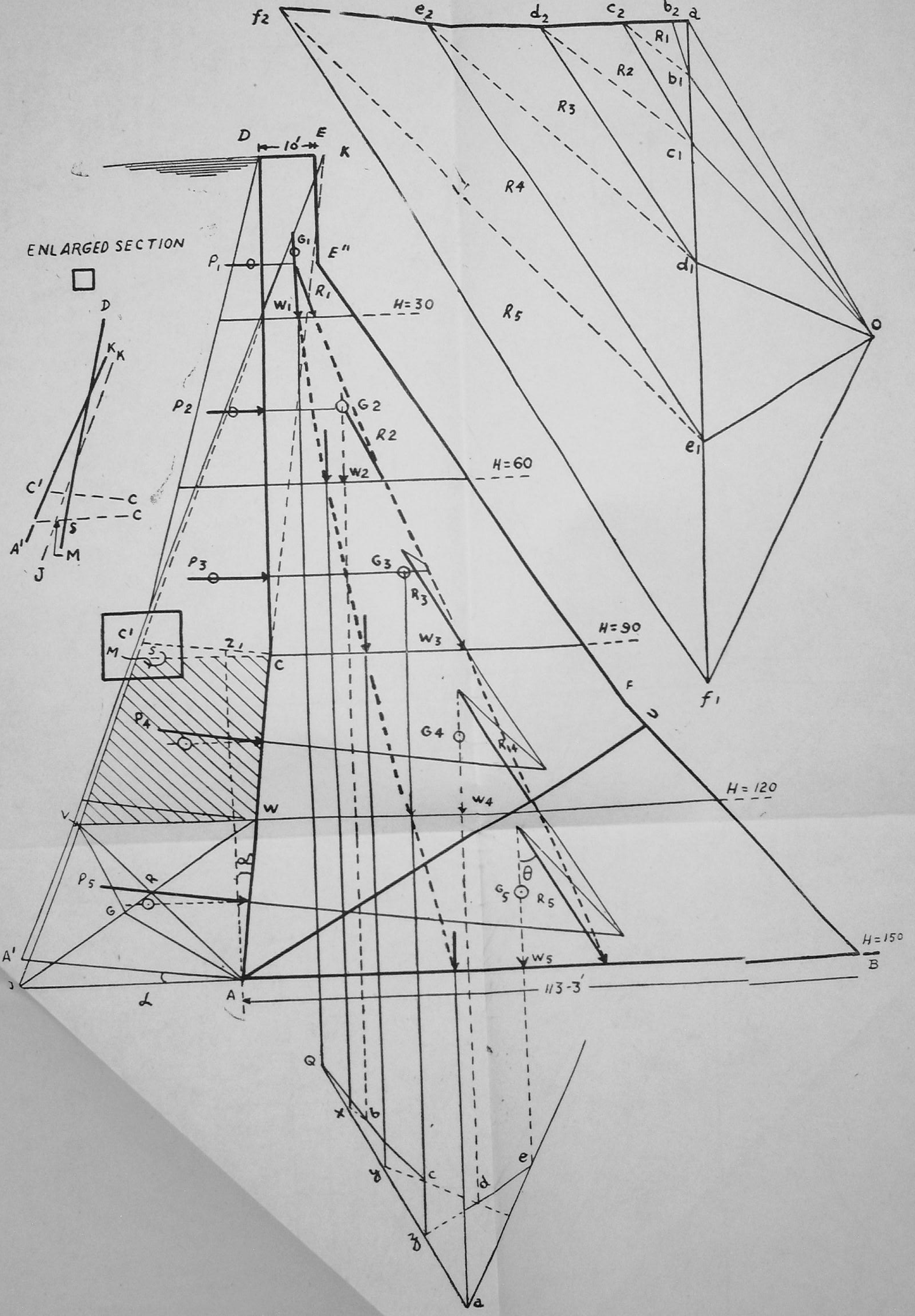

ENLARGED SECTION
D
E
K
10'
E''
G1
P1
W1
R1
H=30
P2
G2
R2
W2
H=60
P3
G3
R3
W3
H=90
C
C'
M
S
Z1
P4
G4
R4
W4
H=120
W
V
P5
G
R
G5
R5
θ
W5
H=150
A'
A
B
113-3'
F
U
Q
x
b
y
c
d
e
a
f2
e2
d2
c2
b2
b1
c1
d1
e1
f1
O
R5
A''
J

Plate I

ENLARGED SECTION
D
E
K
10'
E''
H=30
H=60
H=90
H=120
H=150
113-3'
R1
R2
R3
R4
R5
f2
e2
d2
c2
b2
b1
c1
d1
e1
f1
O
P1
P2
P3
P4
P5
G1
G2
G3
G4
G5
W1
W2
W3
W4
W5
W
C
A
A'
B
F
Q
x
y
z
a
b
c
d
e

Plate I

(i) qa and qb are drawn parallel to oa and ob , the latter cutting G_2 in b,

(ii) bc is drawn parallel to cutting qa in x,

(iii) cd is drawn parallel to Od_1 cutting qa in y,

(iv) de is drawn parallel to Oe_1 cutting qa in z, and

(v) ea is drawn parallel to Of_1 cutting qa in a.

Then a vertical through x would pass through the centre of gravity of W_3 and W_2; a vertical through y would pass through the centre of gravity of W_1, W_2 and W_3, a vertical through z would pass through the centre of gravity of W_1, W_2, W_3 and W_4 and a vertical through a would pass through the centre of gravity of W_1, W_2, W_3, W_4 and W_5 The intersections of these verticals with the bases of the laminae would give the intersections of these bases with the resultant pressure "reservoir empty".

The lines of resultant pressure "reservoir full" R_1, R_2, R_3 etc. are drawn in the dam profile parallel to the similar line in the diagram of forces. is drawn from the intersection of P_1 with G_1 and is produced to intersect P . From this intersection, a line is drawn parallel to the dotted line c_2 b_1 on the diagram of forces being the resultant of P_1 P_2 with W_1. From the point where this intersects G_2 R_2 is drawn on the profile parallel to R_3 on the diagram of forces. Similarly in succession r , R_4, R_5, are drawn on the profile and where these points cut the base lines of the laminae are points of centre of pressure, "Reservoir Full". The curve of pressure "reservoir full" is drawn by joining up the points where R_1 R_2 etc. cut the bases of the corresponding laminae while the curve of pressure 'reservoir empty' is obtained by joining up the points where the vertical through the centre of gravity of all the masonry above the base of each lamina cuts that base.

With regard to the stress induced in the masonry by the resultant R_5 the horizontal base of the dam is not normal to the direction of R_5 but another plane AU is. Consequently the mean stress induced by R_5 is R_5 /AU and not R_5 /AB i.e., the mean unit stress induced in the masonry is R_5/ABCosθ, R_5/AB being the inclination of R_5 to the vertical. On the horizontal base, however, the mean unit pressure would be DCosθ/AB. Having determined the mean unit stress induced in the masonry (or the mean pressure on the foundation as the case may be), and the eccentricity from the centre, the maximum pressure or stress intensity could be graphically computed as follows:*

*The Practical Design of Irrigation Works by BLIGH—pp. 50-51.

Two semi-circles are struck on the base with their centres at the two-third division points and with radius b/3. From e, the point of

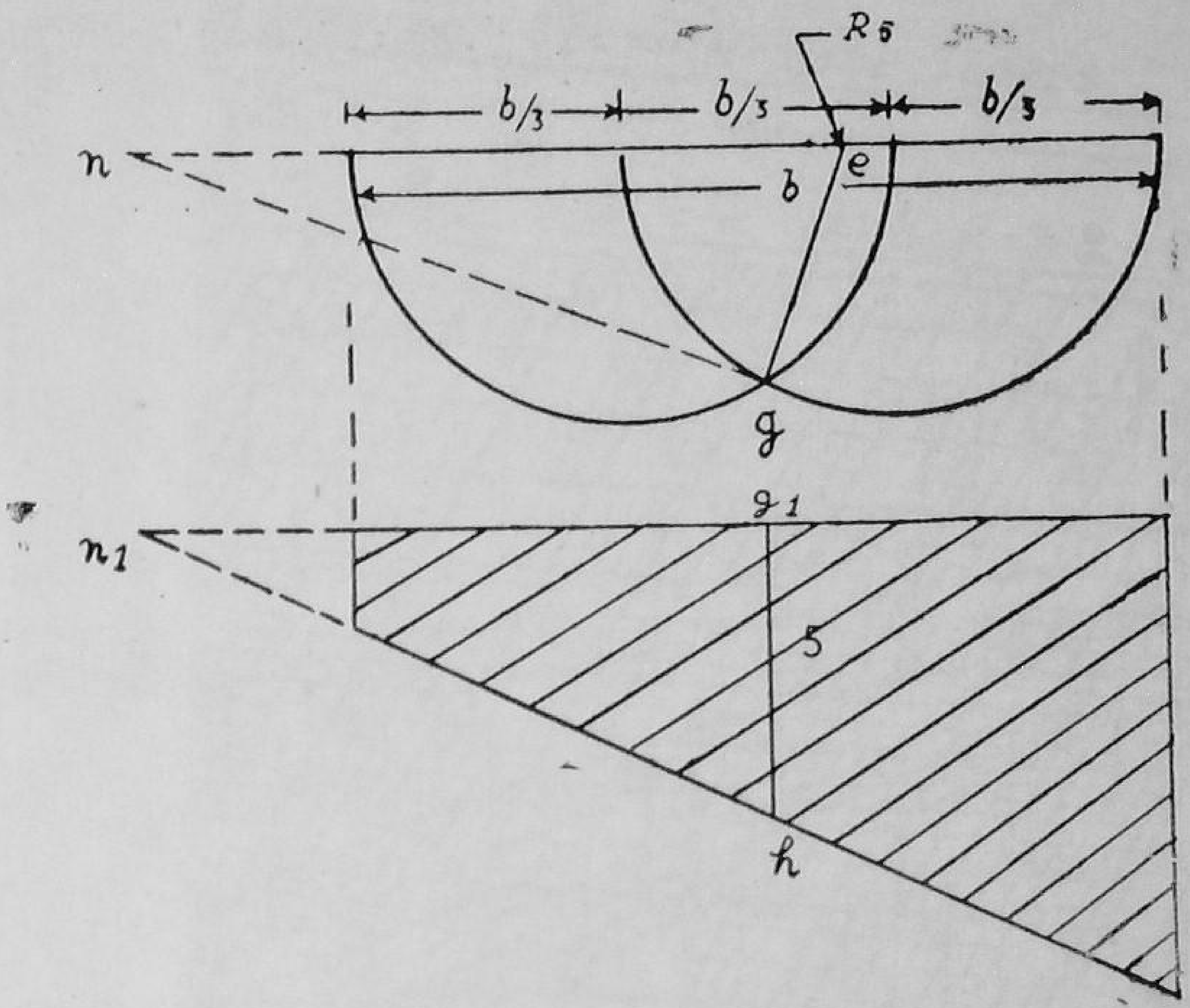


Fig. 9

incidence of R the line eg is drawn to g, the intersection of the semi circles. Again from the point g, a line gn is set off at right angles to eg cutting the base or its continuation at the point n. Below the profile another projection of the base is made. From g, a perpendicular is let fall cutting the new base line in g_1 and its continuation g_1 h is made equal to the mean unit stress S. Another perpendicular is then let fall from n, cutting the new base in n_1. The points n_1 and h are joined and the line continued till it meets the perpendicular from the toe of the base. The hatched trapezoid thus enclosed represents the distribution of pressure.

THE IRRIGATION CHANNEL

The irrigation channel can be designed by Lacey's formulæ *vide* diagram in Plate II.

Losses in Canals.—The losses in canals vary with—

(i) the nature of soil,

(ii) the spring level of the subsoil; and

(iii) the perimeter of the channel.

47 pressure or stress intensity could be graphically computed as follows*:

DISCHARGE & VELOCITY CHART FOR CHANNELS

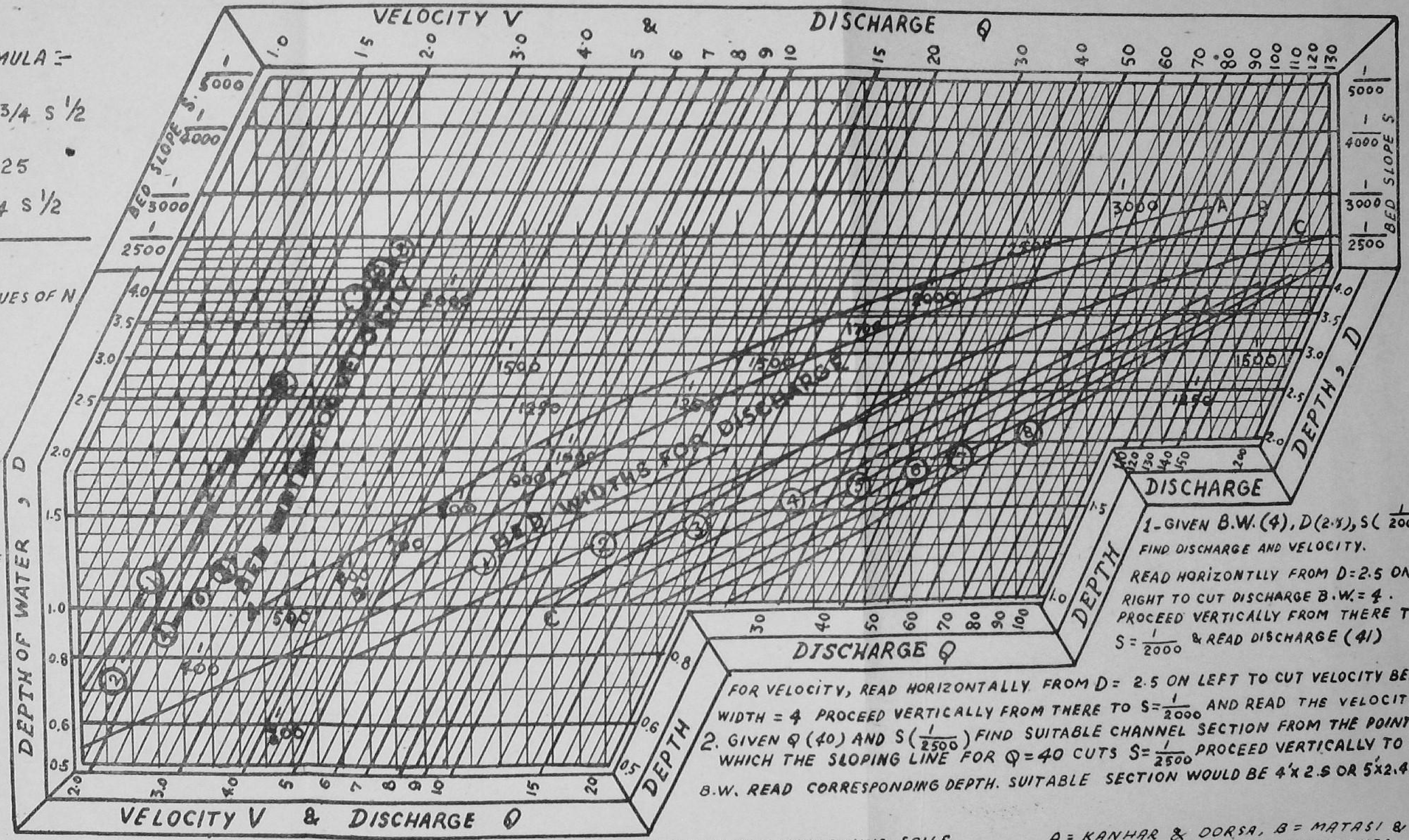
TRAPEZOIDAL - SECTION: SIDE SLOPES 2 to 1

LACEYS' FORMULA :-

$$V = \frac{1.346}{N} R^{3/4} S^{1/2}$$

WITH N = 0.0225

$$V = 60 R^{3/4} S^{1/2}$$



NOTE :- OVER THE RANGE COVERED, THIS CHART APPLIES EQUALLY WELL TO KUTTER'S FORMULA $V = K . R^{1/2} S^{1/2}$ WHERE K IS A COMPLICATED COEFFICIENT.

1. GIVEN B.W. (4), D (2.5), S ($\frac{1}{2000}$), FIND DISCHARGE AND VELOCITY.
READ HORIZONTLLY FROM D = 2.5 ON RIGHT TO CUT DISCHARGE B.W. = 4. PROCEED VERTICALLY FROM THERE TO S = $\frac{1}{2000}$ & READ DISCHARGE (41)
FOR VELOCITY, READ HORIZONTALLY FROM D = 2.5 ON LEFT TO CUT VELOCITY BED WIDTH = 4 PROCEED VERTICALLY FROM THERE TO S = $\frac{1}{2000}$ AND READ THE VELOCITY (1.85
2. GIVEN Q (40) AND S ($\frac{1}{2500}$) FIND SUITABLE CHANNEL SECTION FROM THE POINT AT WHICH THE SLOPING LINE FOR Q = 40 CUTS S = $\frac{1}{2500}$ PROCEED VERTICALLY TO CUT B.W. READ CORRESPONDING DEPTH. SUITABLE SECTION WOULD BE 4'x 2.5 OR 5'x 2.45.

THE LINE A.B.& C MAY BE TERMED THE NON-SCOURING LIMITS OF DISCHARGE IN RELATION TO BED SLOPE IN THE FOLLOWING SOILS A = KANHAR & DORSA, B = MATASI & SOFT MOORUM, C = BHATA & HARD MOORUM. THE POINT OF INTER SECTION OF THE DISCHARGE & BED SLOP SHOULD NOT LIE BELOW THE LINE PERTAINING TO THE SOIL CONCERNED

FOR EXAMPLE A CHANNEL OF NORMAL SECTION CARRYING 10 CUSECS IN KANHAR, WILL TEND TO SCAUR THE BED IF THE SLOP EXCEEDS $\frac{1}{1250}$ IN MATASI THE SLOPE COULD BE STEEPENED TO $\frac{1}{1000}$. A CHANNEL OF NORMAL SECTION, WITH A BED SLOP O $\frac{1}{1500}$ IN HARD MOORUM, CAN CARRY UP TO 40 CUSECS. NO GOVT CHANNAL SHOULD HAVE A SLOPE STEEPER THAN 1 IN 500.

Plate II

pressure or stress intensity could be graphically computed as follows*:

DISCHARGE & VELOCITY CHART FOR CHANNELS

TRAPEZOIDAL - SECTION: SIDE SLOPES 2 to 1

THE LINE A.B.& C MAY BE TERMED THE NON-SCOURING LIMITS OF DISCHARGE IN RELATION TO BED SLOPE IN THE FOLLOWING SOILS A = KANHAR & DORSA, B = MATASI & SOFT MOORUM, C = BHATA & HARD MOORUM. THE POINT OF INTER SECTION OF THE DISCHARGE & BED SLOP SHOULD NOT LIE BELOW THE LINE PERTAINING TO THE SOIL CONCERNED

FOR EXAMPLE A CHANNEL OF NORMAL SECTION CARRYING 10 CUSECS IN KANHAR, WILL TEND TO SCAUR THE BED IF THE SLOP EXCEEDS 1/1250 IN MATASI THE SLOPE COULD BE STEEPENED TO 1/1000. A CHANNEL OF NORMAL SECTION, WITH A BED SLOP O 1/1500 IN HARD MOORUM, CAN CARRY UP TO 40 CUSECS. NO GOVT CHANNAL SHOULD HAVE A SLOPE STEEPER THAN 1 IN 500.

Plate II

As the canals under Minor Irrigation Works are expected to be in small lengths and of small carrying capacity, the losses in transit in such canals would be very small. It would, therefore, generally not be found necessary to make allowances for these. Mr. W. L. Strange suggests an approximation for losses in transit at 4 per cent. per mile of the total discharge in the canal when this discharge is less than 10 cusecs.

CHAPTER VI

THE STREAM WATER—DIVERSION WORKS AND CANALS

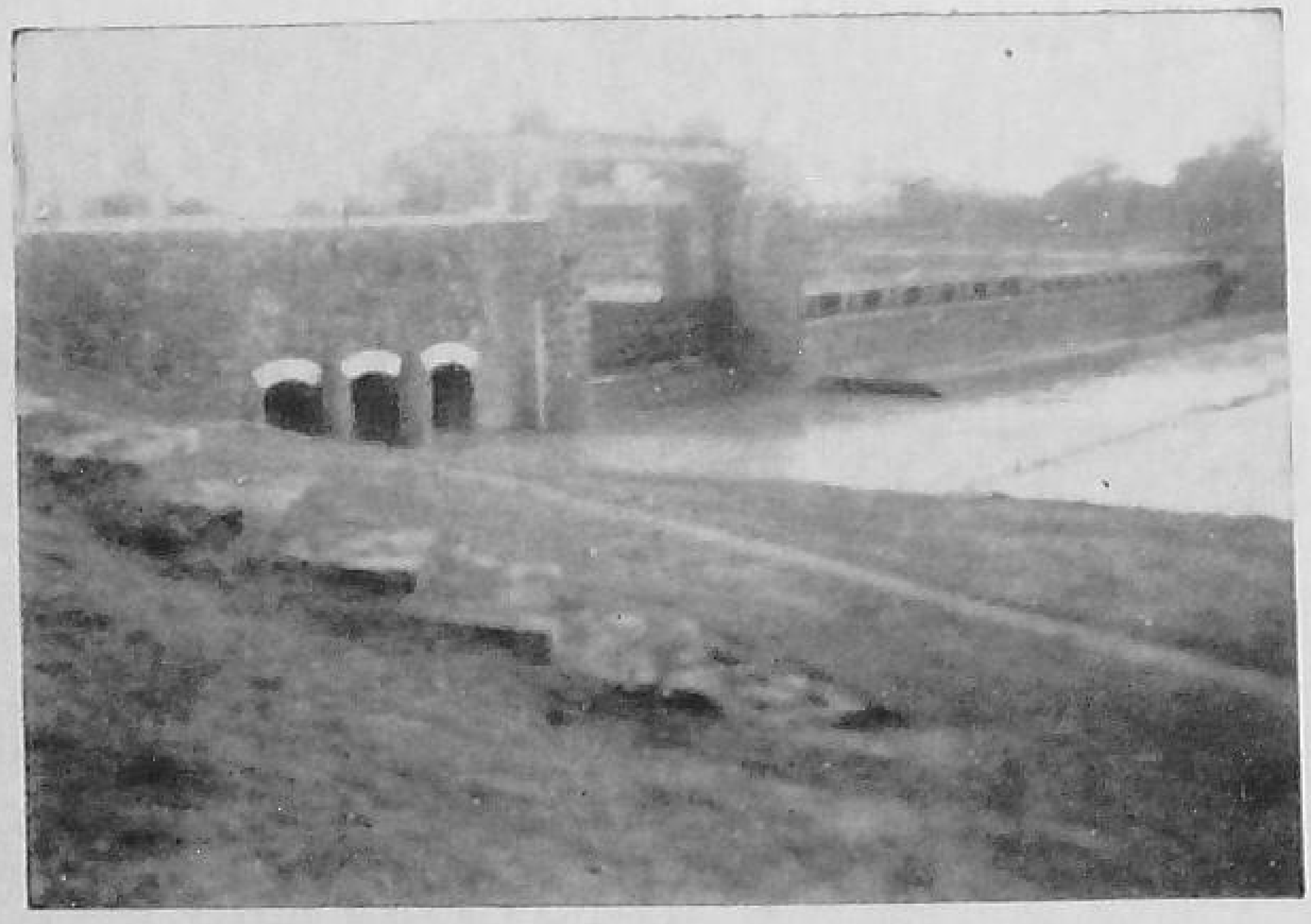
For harnessing the stream water, diversion weir or a regulator is constructed across a stream. The water thus headed up is led through an irrigation channel to a commanding point from where it flows into the fields by gravity. The diversion weir is generally provided with one or more openings, preferably at one end but sometimes in the centre also, fitted with wooden or iron shutters. These are known as "Scouring Sluices" and their function is to prevent silting of the river bed up-stream. As the canal taking off from the weir, is required to carry a fixed discharge only, a suitable sluice control is provided at its head so that during rains, it does not get choked due to the entry of the flood water. Also, sometimes when it is difficult to carry the channel above the high flood contour in its head reach, it is protected from submersion, during floods, by the construction of suitable earthen embankments.

1. **The flow in a stream.**—The irrigation capacity of a diversion work is determined from the actual flow in the stream at the time irrigation is required. If the work is meant for '*kharif*' irrigation, the discharge available in the driest part of this season (*viz.* September or October), will be a measure of its capacity to irrigate. If it is meant for '*rabi*' season, it would be necessary to gauge the post-monsoon flow. The open weather discharge in a stream, does not necessarily depend upon its catchment area. As described later (see Chapter VII), the geology of the catchment is a very important factor. If the sub-soil is porous, the flow ceases immediately after rains, even if the catchment area is big on the other hand, if there are underlying impervious strata or there is continuous supply from a ground water source or some mountain springs or melting snows feed the stream, it would continue to carry a good flow long after the rainy season. The open-season regeneration in a stream thus depends to a great extent, on the geological formation of the tract through which it flows. After the discharge in a stream has been correctly gauged at the time actual irrigation is needed, its irrigation capacity could be determined from the duty adopted for the particular crop (see Chapter III).

2. **Surveys and Location.**—In siting a diversion work, generally the following conditions would be met with:—

(i) Where both the bed of the stream and the adjoining land slope down steeply and it is generally possible to get command from

Madhaborida Anicut in Orissa showing the Canal Headworks



Madhaborida Anicut with scouring sluices on the left and collapsible shutters fixed on the weir crest on the right—Orissa.

a low level diversion weir within a short distance of the head-works. Such sites are commonly met with in the hilly areas and also in the plains at the foot-hills. The Kuhls in the Northern hills or the Dongs in Assam, take off from perennial streams which slope down very steeply. In flatter regions, if the land has a gradient of 1 in 100 to 1 in 500, then also it is possible to site a diversion weir.

(ii) Where land has a less steep slope (1 in 500 to 1 in 1,000). In such cases, it would generally be found necessary to head up water upto the nallah bank level, before any appreciable command could be had. Here the diversion work would take the shape of a regulator or a low weir with collapsible shutters fitted to its crest as, otherwise, if the normal cross-section of the stream is completely blocked, the flood water would have a tendency to out-flank the work during rains.

A weir with collapsible or falling shutters (described later in this chapter) is a fool-proof arrangement and is always to be preferred to a regulator wherever this is found practicable. The disadvantages of a collapsible shutter are—firstly that it is difficult to arrange for its manufacture at short notice. A regulator, on the other hand, has 4 to 5 ft. wide opening which can easily be closed by two rows of planks ("falling boards" or "Karri shutters" as they are called) with earth filled in between (see Fig. 10):—

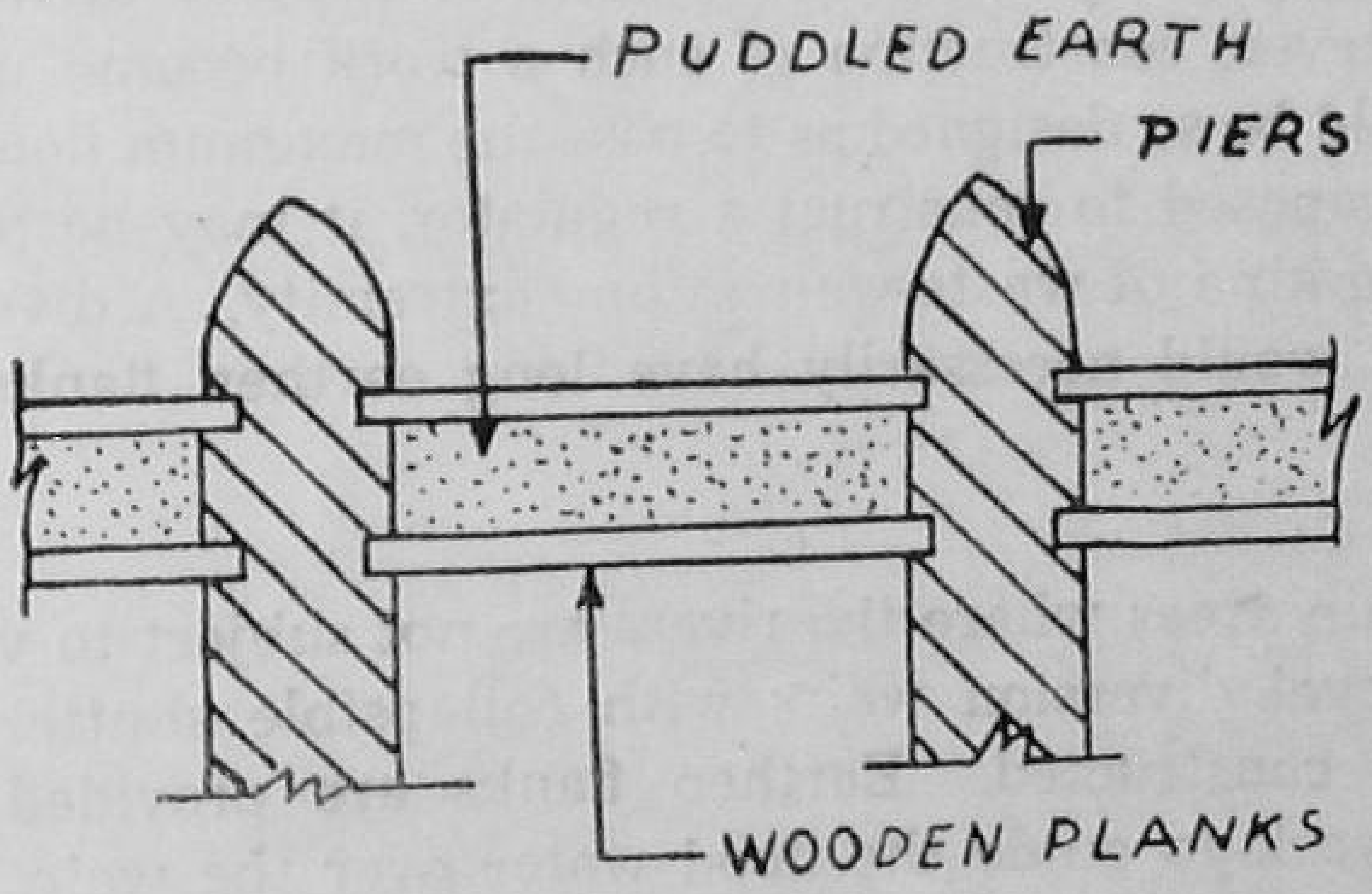


Fig. 10

Villagers find it more convenient to adopt this latter arrangement and often resort to it whenever they want to head up water. The other disadvantage of the collapsible shutter, unless its construction is perfect, is that it cannot stop the leakage of water to the same extent as 'Karri shutters' can do since the puddle earth in the latter arrangement forms a water-tight wall. When, therefore, the flow available, is very small and it is necessary to divert every drop of water into the irrigation channel, the collapsible shutters would not

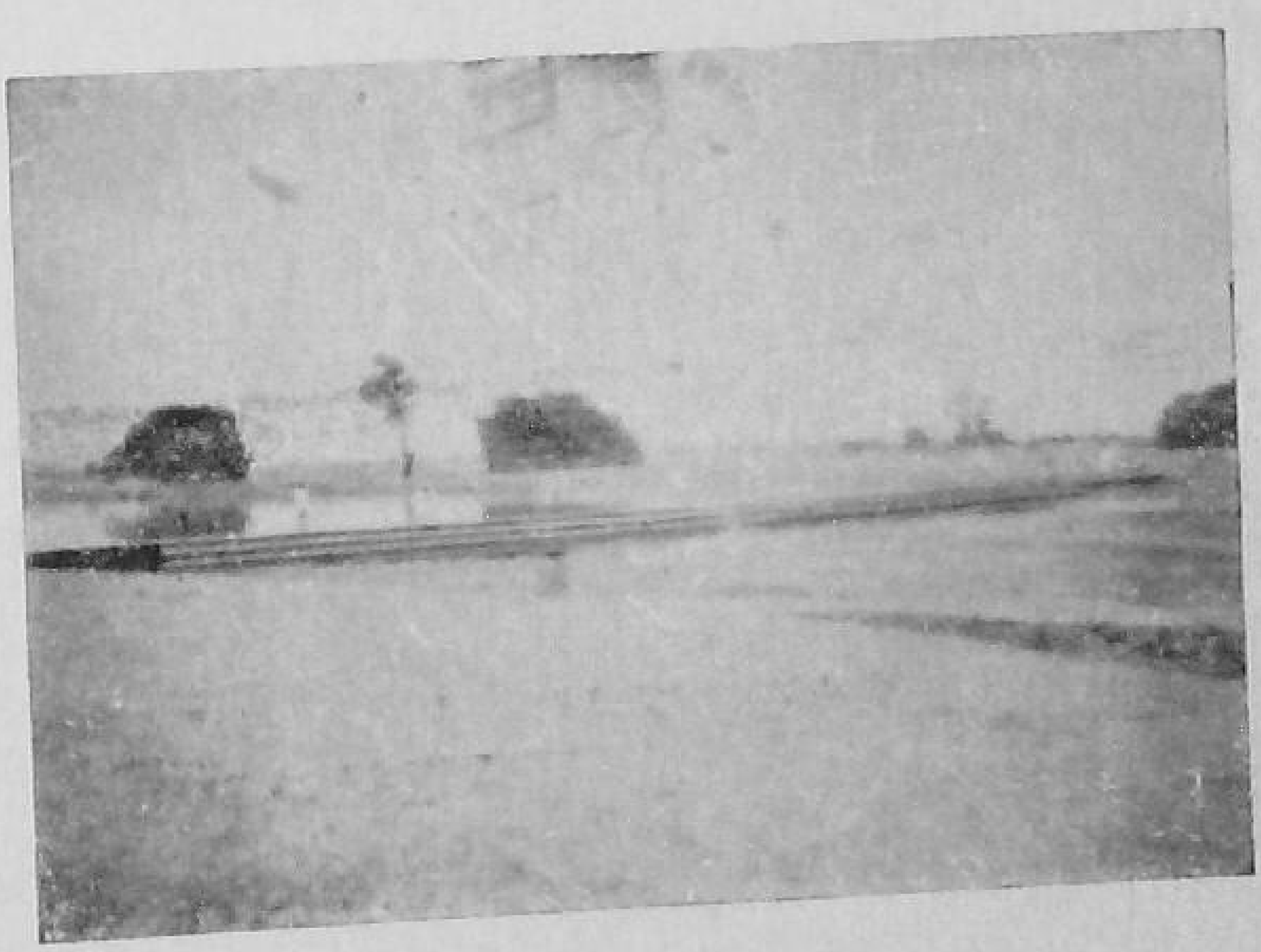
be found to be very effective. These collapsible shutters are generally used on the weir crusts of big irrigation works where large supplies have to be dealt with and leakage of a few cusecs here and there, is not very material. The disadvantages of the regulator arrangement are that the planks cannot be easily removed, when there is a flood, the piers of the regulator offer more obstruction to the water-way than a low level weir and the puddle wall has to be built from year to year making the arrangement somewhat crude and cumbersome.

In the two cases just described, it would be observed that the weir length or the water way to be provided in the regulator is determined from the configuration of the stream *i.e.*, its cross-section and not from considerations of the maximum high flood discharge—in design it would resemble a road cause-way or a submersible bridge. In the case described below, the flood discharge, however, becomes a primary consideration in deciding upon the water-way to be provided in the diversion work.

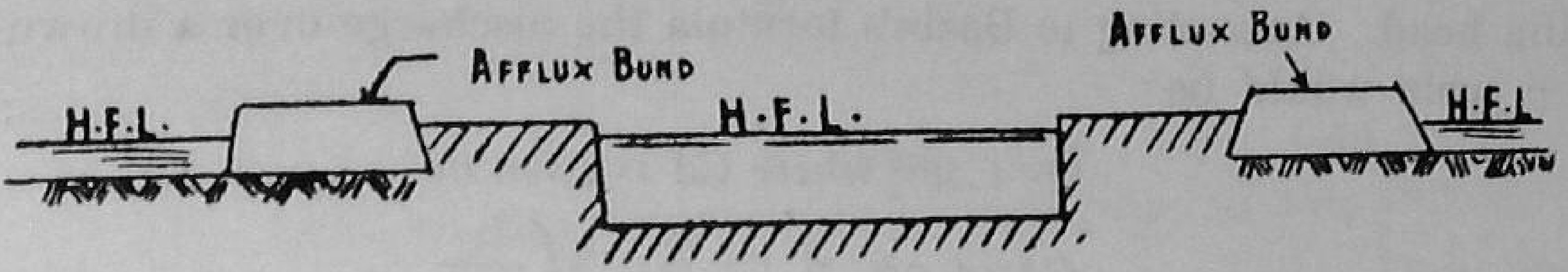
(iii) Where the land may be so flat that unless there is arrangement to head up the water much above the bank level, sometimes upto high flood level, no portion of the land may come under command. In such cases, construction of diversion work should generally be avoided, especially when the stream is subject to heavy floods. Should, however, construction of such a work become unavoidable, the weir could be so designed as to pass the maximum flood discharge or if it is proposed to construct a regulator, it may be necessary to provide some kind of waste-weir at one extremity. A diversion work of this type would necessarily have long earthen flanks on either side.

In the plain areas where the rivers are not subject to very violent floods low level diversion weirs with collapsible shutters could be conveniently constructed. **Earthen flanks are provided on either** side of the work to guide the flood water over the weir. The flood discharge could be calculated either from the cross-section of the nalla and its bed slope or from Dicken's formula $Q=CM^{\frac{3}{4}}$ where Q is the discharge in cusecs, M the catchment area in square miles and C a coefficient. In the plains 'C' could be safely assumed as 600 against 1000 adopted for the hilly country with more rainfall. In U.P. the value of C is taken as 850 for the hilly portion and 200 for the plains under normal conditions. In these areas, the cross-section for the high floods is not restricted and there is general spilling of water on either side into the fields before a flood reaches the dam site—(see Fig. 11).

Pipal Nalla Weir in Bolangir District, Orissa



An Anicut across a river in Madhya Bharat



X

Fig. 11

The weir could, therefore, be safely designed to pass only a portion of the total calculated or observed high flood discharged (50 to 60 per cent. will be sufficient) without involving any risk of the work being out-flanked. The probable discharge passing over the weir having been ascertained, its length could be calculated from the following formula:—

$$L = \frac{Q}{\frac{2}{3}C_1\sqrt{2g}\left\{(H+H_a)^{\frac{3}{2}} - H_a^{\frac{3}{2}}\right\} + \left\{C\sqrt{2g}(H+H_a)^{\frac{1}{2}} \times x\right\}}$$

where L is the length of the weir, Q the flood discharge in cusecs required to be passed over the weir. H is the head in feet due to velocity of approach, x the tail water depth in feet over the crest, H the total depth of water over the crest, c a coefficient for the free overfall=.577 and C a coefficient for the drowned portion=80

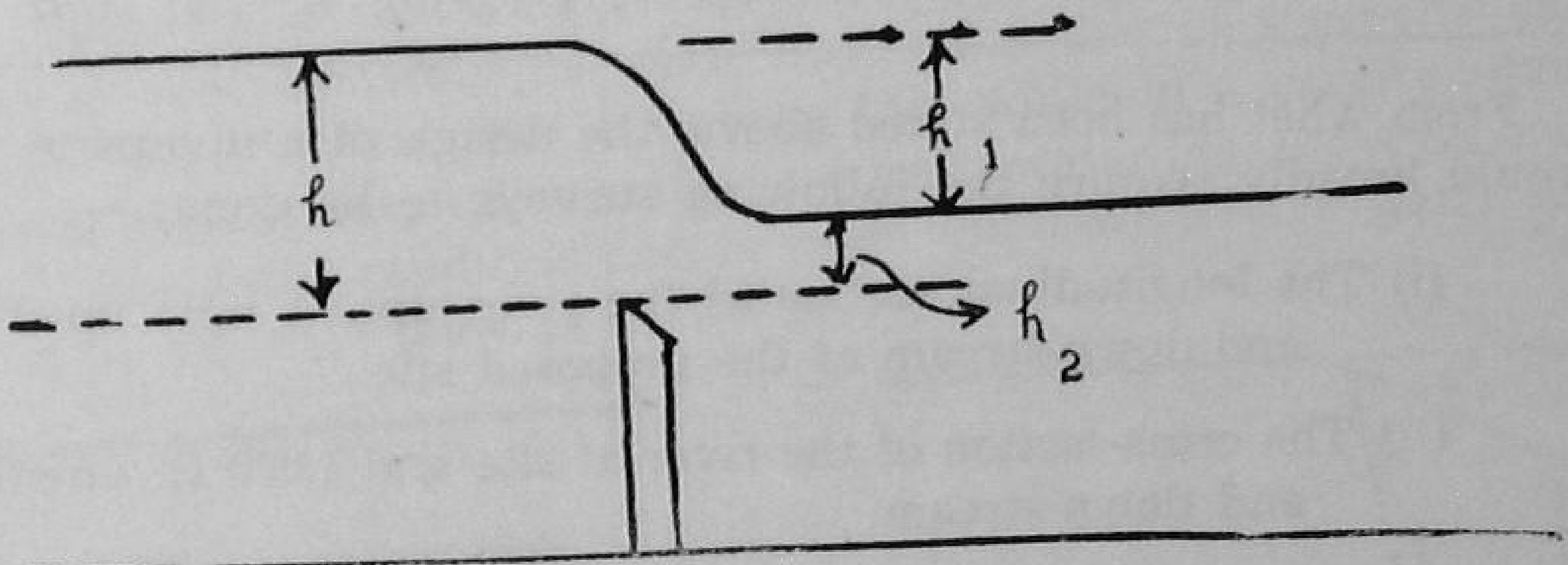


Fig. 12

A much simpler method of calculating the discharge for drowned weirs is by Bazin's Formula*. The fundamental formula for discharge over a weir with a free overfall is

$$Q = LH^{\frac{3}{2}}$$

where C is a coefficient, 1 is the length of the weir, and h the operat-

*Hydraulic Table and Diagrams For Practical Engineers by Capt. Aff. Garre R.E.—P. 17.

ing head. According to Bazin's formula the discharge over a drowned weir would be

$Q = C_d l h^{\frac{3}{2}}$ where Cd is given by

$$C_d = 1.05C\left\{1+\frac{h_2}{186}\right\}\left(\frac{h_1}{\ }\right)^{\frac{1}{3}}$$

where h is the head of water over weir crest upstream, h head of water over weir downstream and h the afflux or difference between water levels upstream and downstream. In the above formula it has been assumed that there is no velocity of approach.

The following table gives the values of $\frac{cd}{c}$ as calculated by the above formula.

Afflux h_1 in feet

I	.2	.4	.6	.8	1.0	1.5	2	2.5	3	4	5
.2	.83	.92	.95	.98	1.00	1.01	1.02	1.03	1.04	1.04	1.05
.4	.74	.84	.89	.94	.96	.98	1.00	1.02	1.03	1 04	1.05
.6	.68	.80	.85	.90	.93	.96	.99	1.01	1.02	1.03	1.05
.8	.64	.76	.81	.87	.90	.94	.97	1.00	1.01	1.02	1 04
1.0	.61	.73	.79	.85	.88	.93	.96	.99	1.10	1.02	1.04
1.5	.55	.67	.74	.80	.84	.90	.94	.97	.99	1.01	1.04
2	.52	.63	.70	.76	.81	.87	.92	.96	.98	1 01	1.04
2.5	.50	.61	.68	.74	.79	.85	.90	95	.98	1.01	1 04
3	.48	.59	.66	.72	.77	.84	.89	.94	.97	1 00	1.04
5	.46	.57	.63	.70	.75	.83	.88	.93	.97	1.00	1.05
5.5	.44	.55	.61	.68	.73	.81	.87	.92	.97	1 01	1.06
7	.42	.53	.69	.66	.71	.79	.85	.91	.97	1 02	1.08

From what has been stated above, the design of a diversion work would broadly require the following surveys to be done:—

(i) The longitudinal section of the river upto 1 mile up-stream and down-stream of the proposed site.

(ii) The cross-section of the river at site and 1,000 ft. up-stream and down-stream.

(iii) The High Flood Level.

(iv) The river flow (to be gauged by a V-notch or some other device) at the time irrigation is required.

(v) Trial pit section in the river bed for ascertaining the nature of foundation available.

(vi) A few lines of fly levels in the area proposed to be commanded by the work.

3. **Design of weirs.**—The weirs are subject to the following forces and their failures may result from one or more of these forces exceed-

ing the safe limit:—

(a) The horizontal thrust of water. The weir should be able to resist the over-turning moment due to this thrust.

(b) The piping action of water. All foundations, specially those sandy in character, are porous to some extent or the other. Water on the up-stream side is at a higher level than that on the down-stream side and has, therefore, a tendency to force its way underneath the foundations of the weir. If the velocity induced in the subsoil flow due to the head of water up-stream exceeds the safe limit, the soil particles underneath would be disturbed and may even move forward. This would endanger the stability of the foundations.

(c) The uplift pressure. The creep water or "the subsoil flow" as it is sometimes termed, is under pressure and has a tendency to lift the floor. If, therefore, the floor is not strong enough to bear the uplift pressure, it might be blown up or fractured.

(d) Kinetic action of the Falling Water. The falling water has a tendency to scour the bed of the river down-stream. Protection works are, therefore, needed to absorb the shock of the falling water and prevent its erosive action.

4. **Stability of the Weir Wall.**—The stability of the weir wall is tested in approximately the same manner as that of a gravity dam (see Chapter V) with the difference that while in a gravity dam, water is present only on one side, in a weir, water is present on both sides. The worst condition occurs when the down-stream water surface is at the crest level as shown below:—

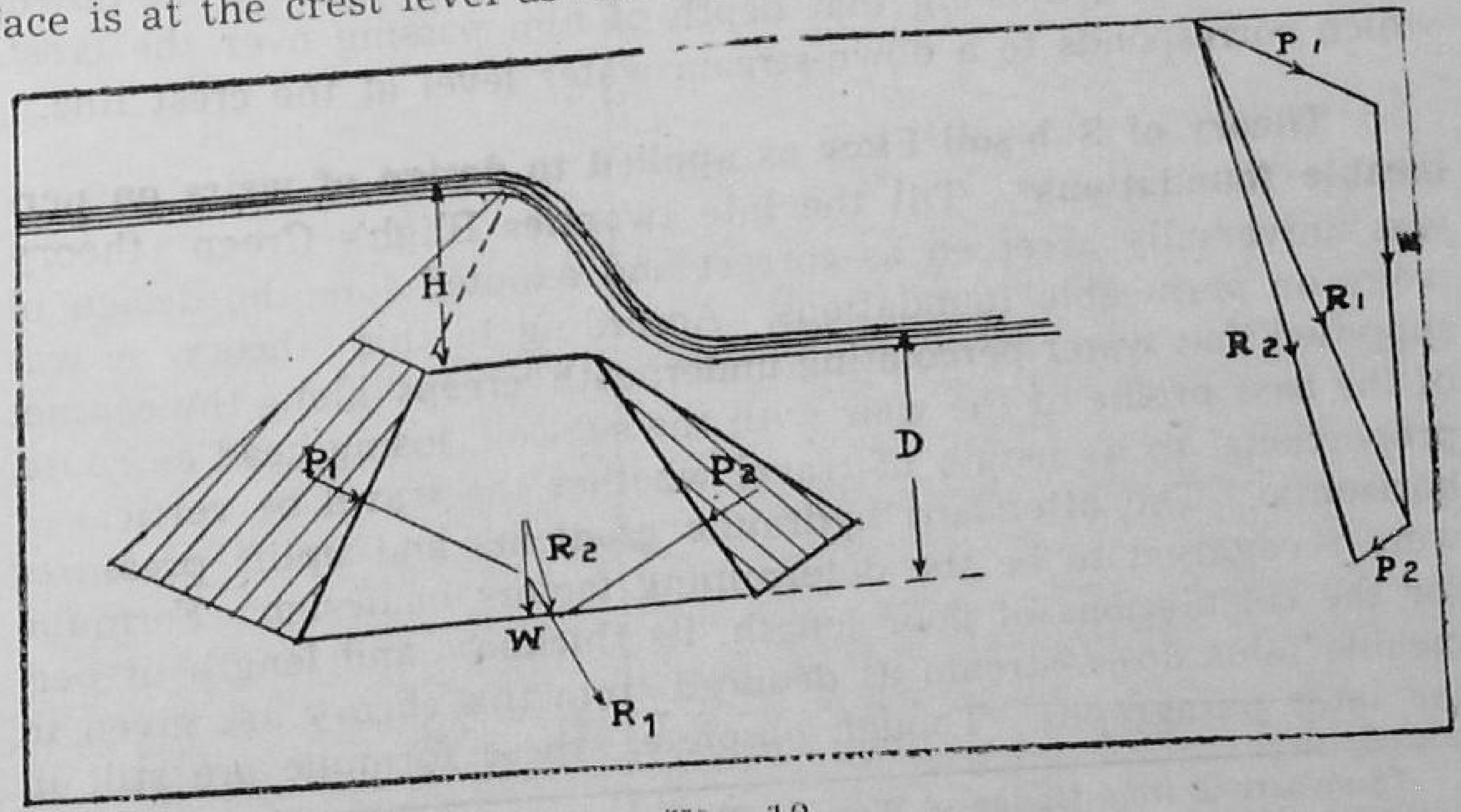


Fig. 13.

For a depth of water "D" (equal to the height of the weir) on the down-stream side, the discharge in the stream is calculated from Chezy's formula.

$V=C\sqrt{RS}$, where V is velocity of water in feet per second. R the hydraulic mean depth and S, the bed slope. The coefficient C is determined either from Kutter's or Bazin's formula, the latter being simpler, according to which

$$C=\frac{157 \cdot 6}{1+\frac{m}{\sqrt{R}}}$$

The value of m varies with the nature of soil being ·38 for rubble masonry, 1·54 for newly dressed earthen channels and 2·34 for ordinary earthen channels. The bed slope 'S' is determined from the longitudinal section of the stream. Having calculated the discharge in the stream corresponding to a depth of water "D", the depth of water H flowing over the weir is calculated. With these water levels, the diagram of forces is drawn as in the case of gravity dams and the resultant R is determined. If it falls within the middle third, the weir will be safe. The resultants are worked out on the supposition that the weir wall rests on a porous formation and is thus subject to loss of weight by floatation. The issoceles profile of the weir is generally found to be economical. The following emperical formulæ will be found useful in fixing the tentative profile of the weir:—

Base width $b=\frac{Hc+d}{\sqrt{P}}$ ....(A)

Crest width $=\sqrt{H}+\sqrt{d}$ subject to a minimum of $\frac{d}{\sqrt{\rho}}$ ....(B)

where Hc is the height of the permanent obstruction above the down-stream floor and d being that depth of film passing over the crest, which corresponds to a down-stream water level at the crest line.

5. **Theory of Sub-soil Flow as applied to design of weirs on permeable foundations*.**—Till the late twenties Bligh's Creep theory was universally accepted as correct and adopted for the design of weirs on permeable foundations. According to this theory, it was supposed that water percolating underneath 'creeps' along the contact of the base profile of the weir with the sub-soil, losing head *en route*, proportional to its length of travel whether the travel be vertical or horizontal. The attendant hydraulic gradients and uplift pressures were recognised to be the determining factors in design. Formulæ for the calculations of floor length, its thickness and length of permeable talus down-stream as deduced from this theory are given in the later paragraphs. Though obsolete, these formulæ are still in

*Summarised from Design of Weir on Permeable Foundation by Khosla C. B.I. Publication No. 12.

use in some of the States specially in Central and Southern India, where the sub-soil is clayey in nature and the effects of percolation are not so marked as in the alluvium of Northern India

During the year 1926-27, failure of certain works in Punjab even though their designs conformed to the principles enunciated by Bligh, aroused a good deal of interest in the subject of flow of water through the sub-soil. Several experiments were performed to study the exact conditions obtaining under a weir structure when water is headed up on the up-stream side. The researches which were mostly carried out by Mr. Khosla of Punjab Irrigation, conclusively proved the error of Bligh's assumption that the length of path of flow had the same effectiveness length for length in reducing the uplift pressure whether it was along the horizontal or vertical. The important conclusions that were arrived at as a result of these experiments were:—

(a) The end sheet pile is much more effective than the horizontal length of floor in reducing the uplift pressure.

(b) The intermediate piles if smaller in length than the outer ones, were ineffective except for local redistribution of pressure.

(c) Undermining of floors started from the tail-end. If the hydraulic gradient at exit was more than the critical gradient for the particular sub-soil, the soil particles would move with the flow of water thus causing progressive degradation of the sub-soil resulting in cavities and ultimate failure.

(d) It was absolutely essential to have a reasonable deep vertical cut off at the down-stream end to prevent undermining.

Subsequently, the investigation on uplift pressures under a weir founded on pervious soil was actively pursued both in this country and the U.S.A. Several experiments were performed with laboratory size model weirs on sand and photographic demonstration of flow of water produced. The idea that the flow of water through the sub-soil was not in horizontal and vertical straight lines but was in stream lines and could be likened to the flow of electricity through a conductor gained ground. Full size experiments were, however, performed for the first time on the extension of the Panjnad weir in Punjab which confirmed the conclusions arrived at earlier. The fact that the flow of water through the sub-soil is in stream lines and is, therefore, susceptible of mathematical treatment was established beyond doubt. The theory evolved on the basis of these assumptions, known after its author as "Khosla's Theory of Sub-soil Flow" is now universally accepted and adopted for the design of weirs on permeable foundations.

The two theories and their application in the design of weirs are described in some detail in the subsequent paragraphs.

6. **Bligh's Creep Theory:**

(i) **Piping Action.**—According to this theory the piping action below the foundations is dependent upon the head of water and the permeability of the soil. If the soil is porous or the head of water is great, conditions will generally favour the flow of sub-soil water with a high velocity and if this water finds a suitable exit down-stream, sand or soil will have a tendency to be blown out resulting in the undermining of foundations. In designing a weir, this is guarded against by providing sufficient length of flooring which is impervious to water. If the floor is long enough, the entire head of water would be lost in its passage through the sub-soil below the floor and water would just trickle out without any danger of the soil underneath the foundations, being disturbed. This length is known as the creep length and is given by cH, where H is the operating head of water and "c" a coefficient dependent upon the soil. The values of "c" for different kinds of soils are given below.—

(a) River beds of light silt and mud

$c=18$

(b) Fine micacious sand as in the Himalayan rivers

$c=15$

(c) Coarse Grain sand as in Central and South India

$c=12$

(d) Boulders or shingle and gravel and sand mixed

$c=9$ to 5

(e) Ordinary soil

$c=5$

The length cH is provided into two parts and sometimes into three parts also. One is the pucca impermeable flooring down-stream of the work, the second is the puddle lining up-stream with a covering of light soil, murrum or boulders, and thirdly in the form of sheet piles or curtain walls.

The path of water according to this theory is assumed to be along the bottom of the floor as shown by arrows in the diagram, the total creep length cH would be equal to $W+b+2c+L$, and the hydraulic gradient line of the creep water would be somewhat as shown in fig. 14 (assuming the value of 15 for c).

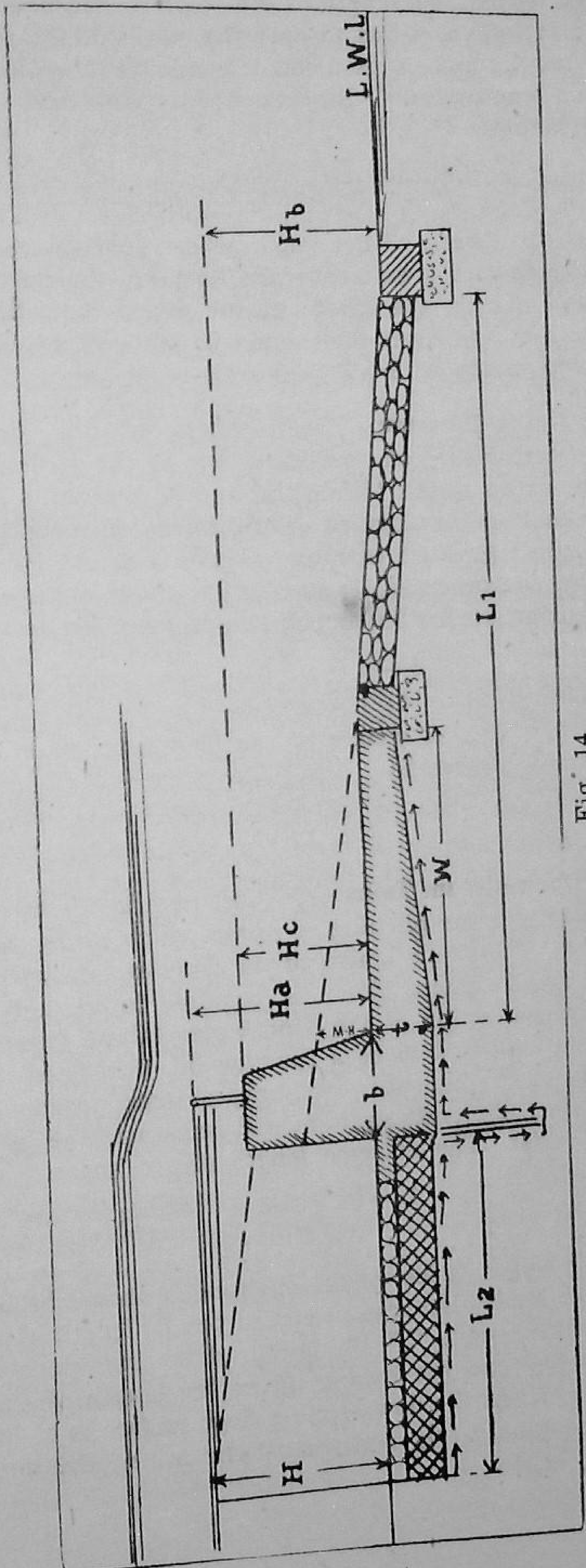


Fig. 14.

As stated earlier, the length of the down-stream floor is governed by another factor *i.e.* it has to bear the shock of the falling water. The following formula is used for determining the length of the down-stream impermeable flooring which takes into consideration both these factors:—

$$\text{“}W = 4c\sqrt{\frac{Ha}{13}} \qquad \text{(c)”}$$

where W is the length of the impermeable flooring and Ha is the height of the shutter crest above the floor *i.e.* the operating head. Having found out W, the length of the puddle flooring up-stream and the depth of the sheet piles could be suitably adjusted so that the total creep length is 15 c.

(ii) **The Uplift Pressure.**—The thickness of the down-stream flooring is determined from considerations of the hydrostatic pressure of the creep water. From the line of Hydraulic gradient, it would be seen that the floor at 'A' should be of such thickness as to bear the uplift pressure corresponding to a height H-h of water. Since the floor is immersed, its specific gravity is reduced by 1 and the t thickness at the particular point could be given by

$$t = \frac{H-h}{\rho-1}$$

In actual practice the weight of the floor is made at least one-third more and t is given by

$$\text{“}t = \frac{4}{3} \times \frac{H-h}{\rho-1} \qquad \text{(D)”}$$

(iii) **The Down-stream Talus.**—The last item in the design of the weir is the packed stone pitching or riprap which forms the talus of the weir body. The whole length of the down-stream apron including the impermeable pucca floor is required to protect the work against the erosive action of the flood water falling from a height. It is. therefore, supposed to be a function of the flood discharge and the height of the permanent masonry obstruction above the low water level (since the crest shutters will remain in the falling position during floods). The extreme length of the apron is given by

$$L_2 = 10C\sqrt{\frac{Hb}{10}}\sqrt{q} \qquad \text{(E)}$$

where H is the height in feet of the obstruction above the low water level and q, the flood discharge in cusecs per ft. length of the weir. From the extreme length "L", the length of the impermeable flooring W is deducted and the balance gives the length of the packed stone pitching. When the down-stream floor surface is at low water level, H_b will be equal to H_c and when there are no shutters, Ha will be equal to H_a.

7. **Khosla's theory***.—According to this theory, water in its flow through the subsoil under a weir or other hydraulic structure follows a stream line path as shown below:—

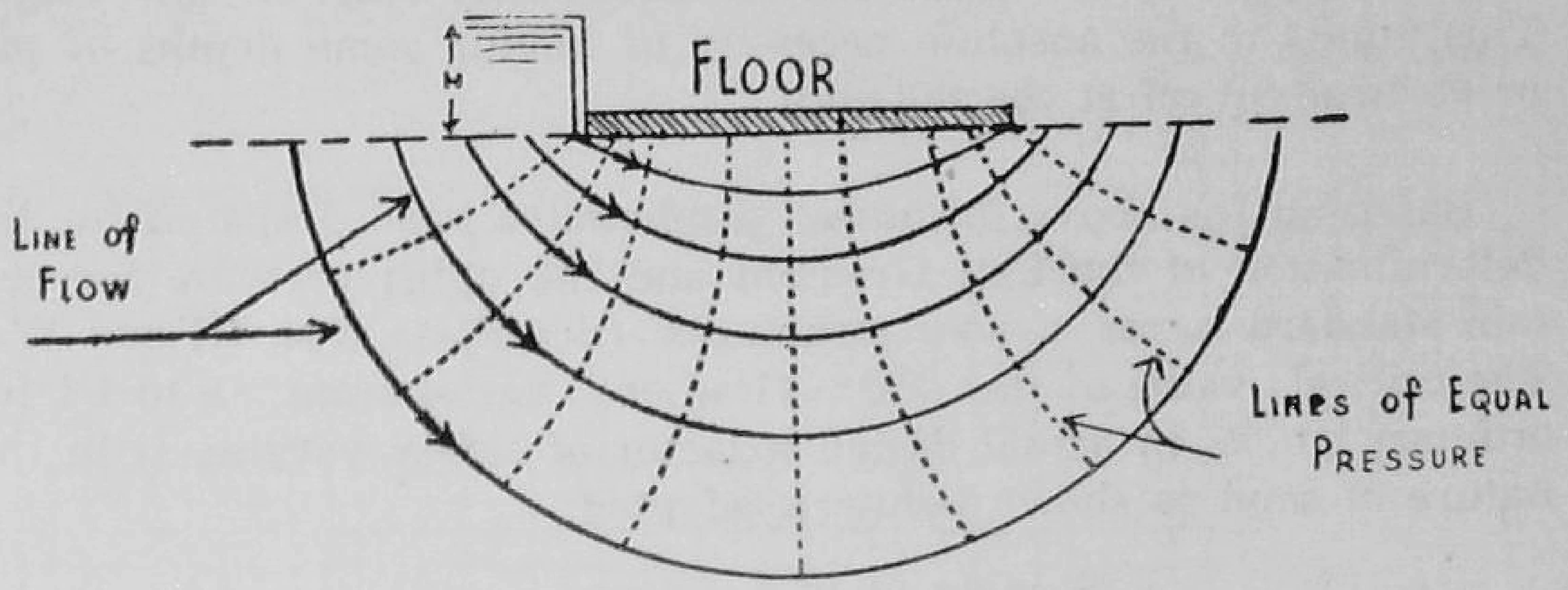


Fig. 15.

In its passage, water loses head gradually till at the exit, the head is reduced to zero. Thus at each point under the weir-floor, water has a residual pressure and a certain pressure gradient, *i.e.*, the rate at which the head is being dissipated. The pressure gradient at any point is a measure of the force which the subsoil exerts in the direction of flow. Thus there are two forces which the weir has to withstand—firstly, the residual pressure which tends to lift up the weir floor and secondly, the pressure gradient or the force acting in the direction of the flow. This latter is of no consequence except at the tail-end where the water emerges from the subsoil. If at this end, the upward force of water is in excess of the restraining force of the sub-soil *viz.*, its weight and internal friction, the surface soil would be lifted up followed by progressive disruption further down. This pressure gradient at the tail-end is termed as "Exit Gradient". The limiting value of the Exit Gradient is sometimes called the "Floatation Gradient" or the "Critical Gradient" or the "Bursting Gradient". The mathematical determination of the Exit Gradient and the uplift pressure for any particular foundation profile of weir, with or without sheet piles, involves the solution of very complicated equations, which is not within the scope of this Manual. A significant feature of the formulæ for Exit Gradient for all standard forms of foundation profile, is that the depth of the down-stream pile "d" occurs as denominator so that the greater the value of "d" in relation to the operating head 'H', the less the Exit Gradient and the more stable the work. This is the fundamental principle to be kept in mind for designs. If only a simple floor is provided without a down-stream pile line, "d" would be equal to zero and the Exit Gradient would become infinity. Such a structure would be unstable and undermining must start at the tail-end. From the formulæ

*Extracted from "Design of Weirs on Permeable Foundation" By Khosla C.B. Publication No. 12.

deduced for such a profile, it would, however, be noticed that the infinite gradient rapidly falls off and soon gets to a safe figure. But all the same, a certain length is subjected to gradient in excess of the critical value and trouble must necessarily start in this length. This points to the absolute necessity of having some depths of pile or vertical cut off at the tail-end.

Based on the above formulae, graphs have been prepared for the determination of the Exit Gradient and the uplift pressure for certain standard forms of weir structures. (See Plate Nos. III and IV.) The critical value of the Exit Gradient varies from ·9 to 1·1 for ordinary sands. In actual design a factor of safety varying with the nature of sand as shown below is adopted:—

Shingle—4 to 5.
Coarse sand—5 to 6.
Fine sand—6 to 7.

For any complex weir section, the "Method of Independent Variables" is followed for determining the Exit Gradient and the uplift pressure. In this method, any complex weir section is split up into its elementary standard forms—the entire length of the floor with any one of the pile lines making up one such form. Each elementary form is then treated as independent of the others. The pressures at the key points are then read from the graphs. These key points are the junction points of the floor and the pile line of that particular form, the bottom point of that pile-line and the bottom corners in the case of a depressed floor. The readings at the junction points are then corrected for —

(a) the mutual interference of piles;
(b) Distance between the two piles,
(c) the slope of the floor.

The mutual interference of piles is given by a simple formula—

$$\frac{\sqrt{D}}{b'} \times \frac{d+D}{b}$$

where C=The correction to be applied as percentage of head.

b′ = Distance between the two piles.

D=The depth of pile, whose influence has to be determined on the neighbouring pile of depth (d).

d=Depth of pile, on which the effect of pile (D) is sought to be determined.

and b=Total floor length.

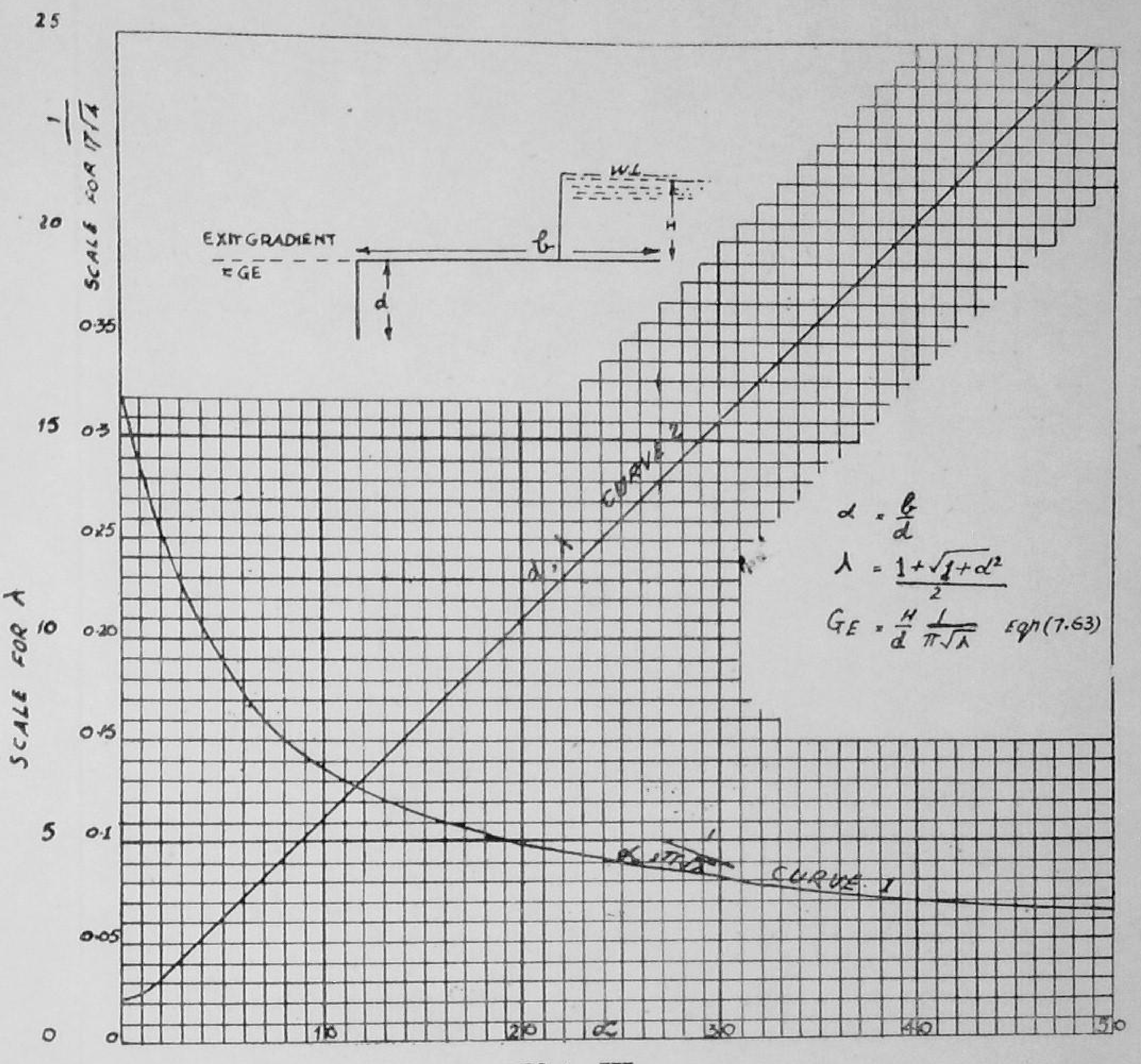
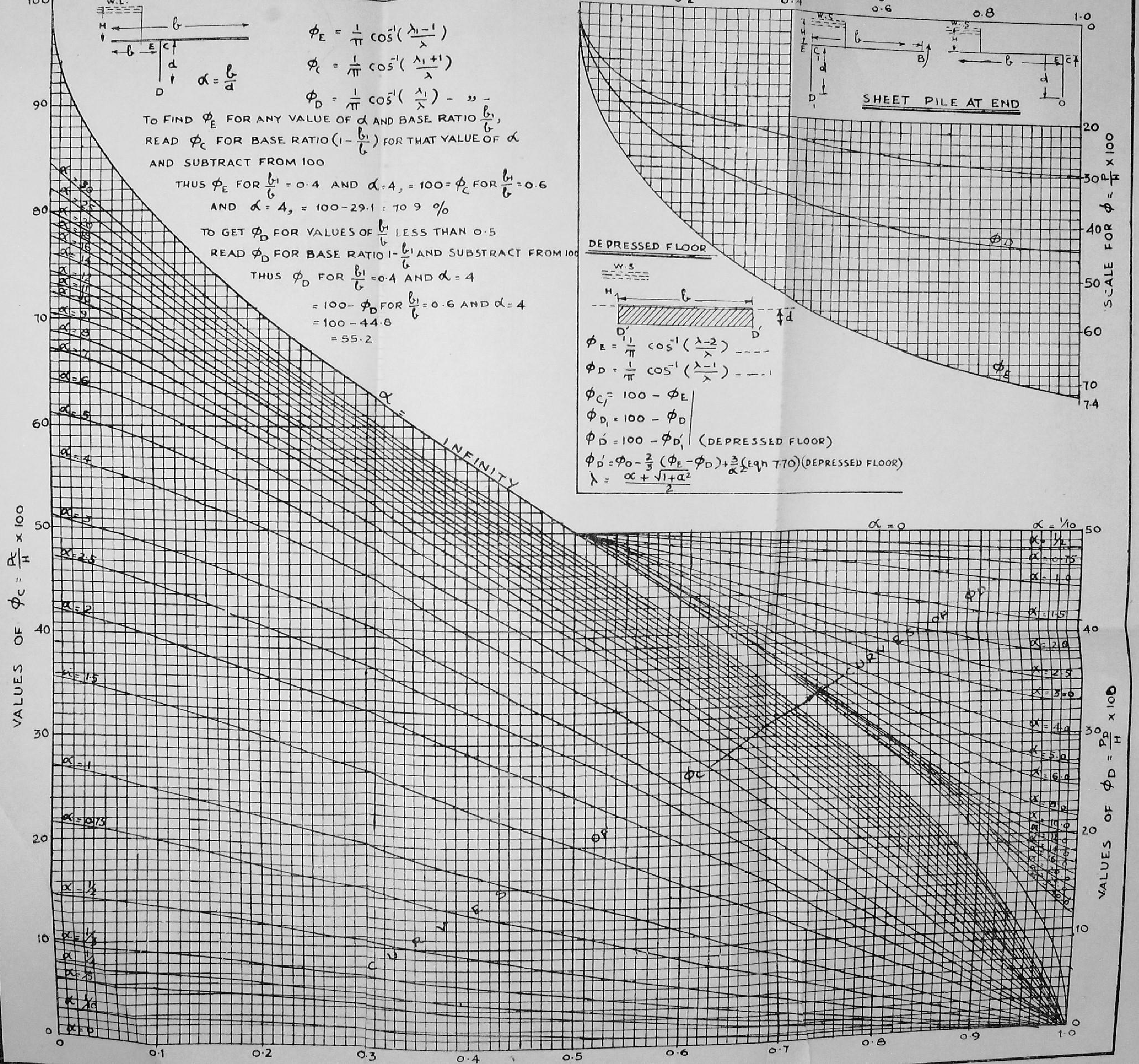
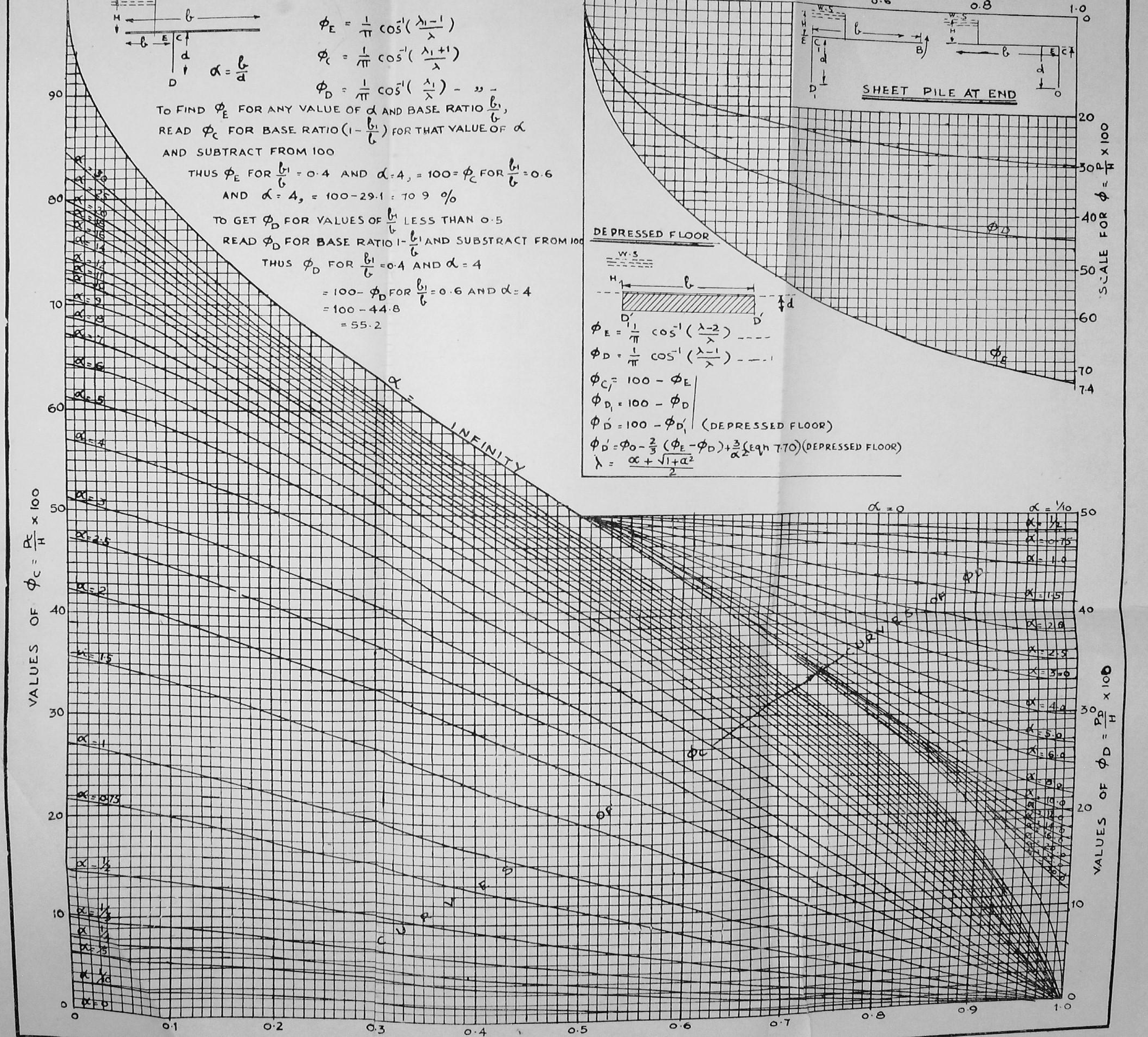

SCALE FOR $\frac{1}{\pi\sqrt{\lambda}}$
SCALE FOR λ
EXIT GRADIENT
= GE
W.L.
b
d
H
CURVE 2
α, λ
CURVE 1
$\alpha = \frac{b}{d}$
$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$
$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$ Eqn (7.63)
α
0
5
10
15
20
25
0.05
0.1
0.15
0.20
0.25
0.3
0.35
10
20
30
40
50

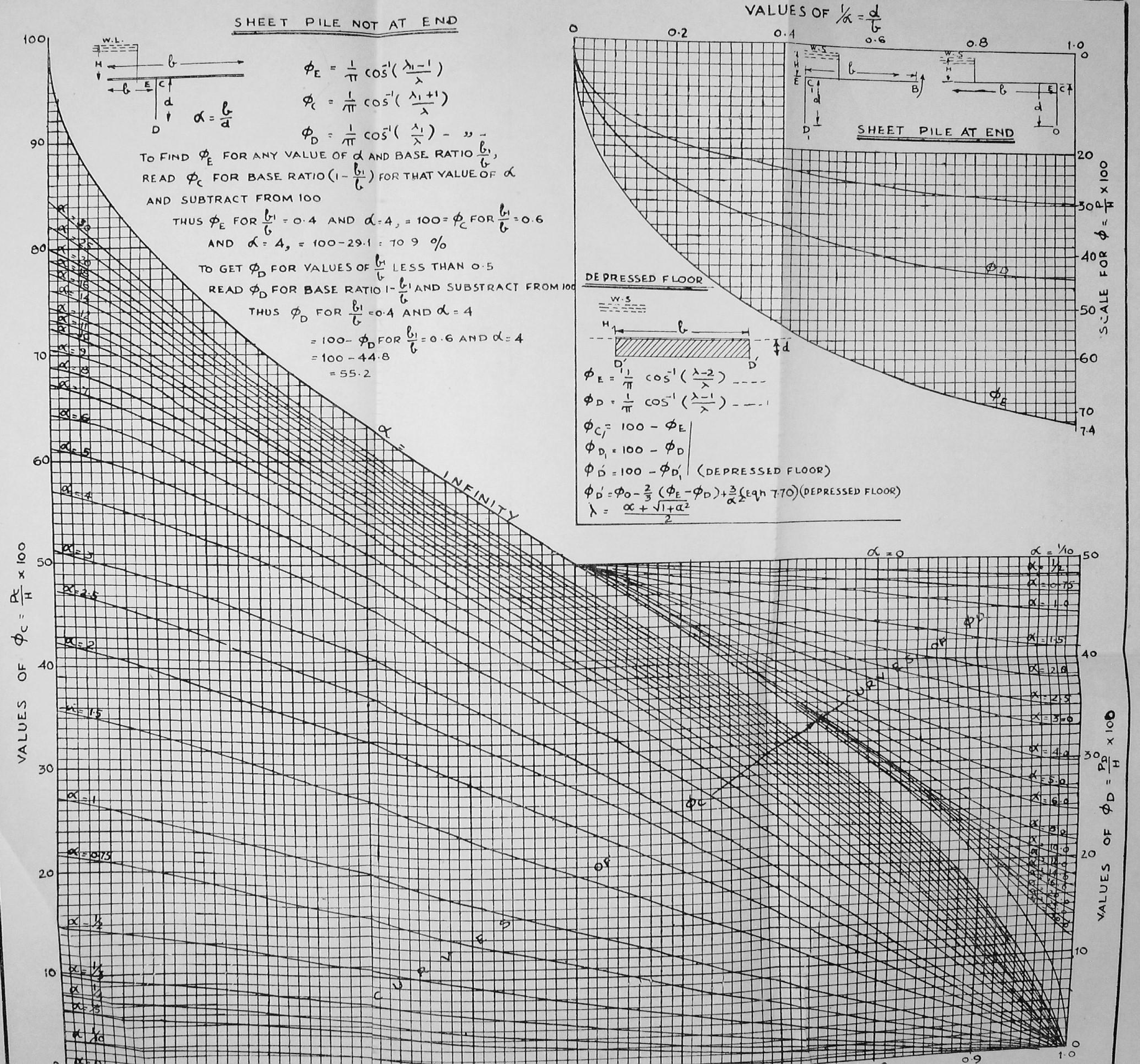
Plate III

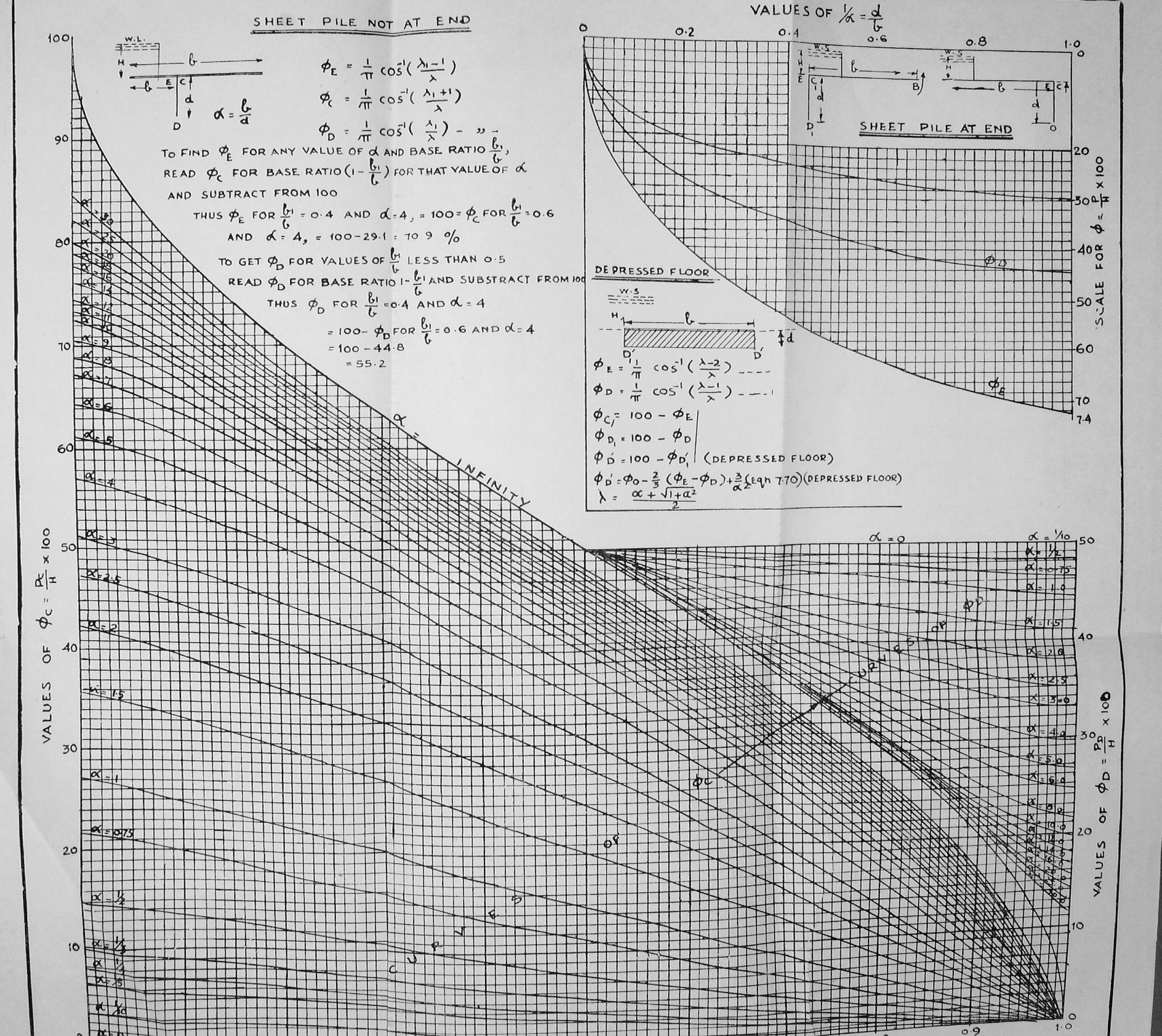


W.L.
H
b
E
C
d
D
$\alpha = \frac{b}{d}$
$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$
$\phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$
$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$ - „ -
TO FIND ϕ_E FOR ANY VALUE OF α AND BASE RATIO $\frac{b_1}{b}$,
READ ϕ_C FOR BASE RATIO $(1-\frac{b_1}{b})$ FOR THAT VALUE OF α
AND SUBTRACT FROM 100
THUS ϕ_E FOR $\frac{b_1}{b} = 0.4$ AND $\alpha = 4$, = 100 = ϕ_C FOR $\frac{b_1}{b} = 0.6$
AND $\alpha = 4$, = 100 - 29.1 = 70 9 %
TO GET ϕ_D FOR VALUES OF $\frac{b_1}{b}$ LESS THAN 0.5
READ ϕ_D FOR BASE RATIO $1-\frac{b_1}{b}$ AND SUBSTRACT FROM 100
THUS ϕ_D FOR $\frac{b_1}{b} = 0.4$ AND $\alpha = 4$
= 100 - ϕ_D FOR $\frac{b_1}{b} = 0.6$ AND $\alpha = 4$
= 100 - 44.8
= 55.2
α = INFINITY
W.S
B
SHEET PILE AT END
ϕ_D
ϕ_E
SCALE FOR $\phi = \frac{P}{H} \times 100$
DEPRESSED FLOOR
W.S
H
b
d
D'
D'
$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$
$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$
$\phi_{C_1} = 100 - \phi_E$
$\phi_{D_1} = 100 - \phi_D$
$\phi_{D'} = 100 - \phi_{D'_1}$ (DEPRESSED FLOOR)
$\phi_{D'} = \phi_D - \frac{2}{3}(\phi_E - \phi_D) + \frac{3}{\alpha^2}$ (eqn 7.70) (DEPRESSED FLOOR)
$\lambda = \frac{\alpha + \sqrt{1+\alpha^2}}{2}$
VALUES OF $\phi_C = \frac{P_C}{H} \times 100$
VALUES OF $\phi_D = \frac{P_D}{H} \times 100$
CURVES OF ϕ_C
CURVES OF ϕ_D
α = 0
α = 1/10
α = 1/2
α = 0.75
α = 1.0
α = 1.5
α = 2.0
α = 2.5
α = 3.0
α = 4.0
α = 5.0
α = 6.0
α = 8.0
α = 10.0
α = 1/3
α = 1/4
α = 1/5
α = 1/10
α = 1
α = 1.5
α = 2
α = 2.5
α = 3
α = 4
α = 5
α = 6
α = 7
α = 8
α = 9
α = 10
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0



$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$
$\phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$
$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$ - „ -
$\alpha = \frac{b}{d}$
TO FIND ϕ_E FOR ANY VALUE OF α AND BASE RATIO $\frac{b_1}{b}$, READ ϕ_C FOR BASE RATIO $(1-\frac{b_1}{b})$ FOR THAT VALUE OF α AND SUBTRACT FROM 100
THUS ϕ_E FOR $\frac{b_1}{b}$ = 0.4 AND α = 4, = 100 = ϕ_C FOR $\frac{b_1}{b}$ = 0.6
AND α = 4, = 100 - 29.1 = 70.9 %
TO GET ϕ_D FOR VALUES OF $\frac{b_1}{b}$ LESS THAN 0.5
READ ϕ_D FOR BASE RATIO $1-\frac{b_1}{b}$ AND SUBSTRACT FROM 100
THUS ϕ_D FOR $\frac{b_1}{b}$ = 0.4 AND α = 4
= 100 - ϕ_D FOR $\frac{b_1}{b}$ = 0.6 AND α = 4
= 100 - 44.8
= 55.2
SHEET PILE AT END
SCALE FOR $\phi = \frac{P}{H} \times 100$
ϕ_D
ϕ_E
DEPRESSED FLOOR
W.S
$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$ ----
$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$ -.-.-
$\phi_{C_1} = 100 - \phi_E$
$\phi_{D_1} = 100 - \phi_D$
$\phi_{D'} = 100 - \phi_{D'_1}$ (DEPRESSED FLOOR)
$\phi_{D'} = \phi_D - \frac{2}{3}(\phi_E - \phi_D) + \frac{3}{\alpha^2}$ (eqn 7.70) (DEPRESSED FLOOR)
$\lambda = \frac{\alpha + \sqrt{1+\alpha^2}}{2}$
α = INFINITY
α = 0
CURVES OF ϕ_D
CURVES OF ϕ_C
VALUES OF $\phi_C = \frac{P_C}{H} \times 100$
VALUES OF $\phi_D = \frac{P_D}{H} \times 100$


SHEET PILE NOT AT END
W.L.
H
b
E
C
d
D
α = b/d
$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$
$\phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$
$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$
TO FIND ϕ_E FOR ANY VALUE OF α AND BASE RATIO $\frac{b_1}{b}$,
READ ϕ_C FOR BASE RATIO $(1 - \frac{b_1}{b})$ FOR THAT VALUE OF α
AND SUBTRACT FROM 100
THUS ϕ_E FOR $\frac{b_1}{b} = 0.4$ AND α = 4, = 100 − ϕ_C FOR $\frac{b_1}{b} = 0.6$
AND α = 4, = 100 − 29.1 = 70.9 %
TO GET ϕ_D FOR VALUES OF $\frac{b_1}{b}$ LESS THAN 0.5
READ ϕ_D FOR BASE RATIO $1 - \frac{b_1}{b}$ AND SUBSTRACT FROM 100
THUS ϕ_D FOR $\frac{b_1}{b} = 0.4$ AND α = 4
= 100 − ϕ_D FOR $\frac{b_1}{b} = 0.6$ AND α = 4
= 100 − 44.8
= 55.2
VALUES OF 1/α = d/b
0
0.2
0.4
0.6
0.8
1.0
W.S
H
E
C
D
d
b
B
SHEET PILE AT END
SCALE FOR $\phi = \frac{P}{H} \times 100$
20
30
40
50
60
70
7.4
ϕ_D
ϕ_E
DEPRESSED FLOOR
W.S
H
b
d
D'
$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$ ----
$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$ —·—·
$\phi_{C_1} = 100 - \phi_E$
$\phi_{D_1} = 100 - \phi_D$
$\phi_{D'} = 100 - \phi_{D'_1}$ (DEPRESSED FLOOR)
$\phi_{D'} = \phi_D - \frac{2}{3}(\phi_E - \phi_D) + \frac{3}{\alpha^2}$ (Eqn 7.70) (DEPRESSED FLOOR)
$\lambda = \frac{\alpha + \sqrt{1 + \alpha^2}}{2}$
α = INFINITY
VALUES OF $\phi_C = \frac{P_C}{H} \times 100$
100
90
80
70
60
50
40
30
20
10
α = 20
α = 18
α = 16
α = 14
α = 12
α = 11
α = 10
α = 9
α = 8
α = 7
α = 6
α = 5
α = 4
α = 3
α = 2.5
α = 2
α = 1.5
α = 1
α = 0.75
α = 1/2
α = 1/3
α = 1/4
α = 1/5
α = 1/10
α = 0
CURVES OF ϕ_C
CURVES OF ϕ_D
α = 0
α = 1/10
α = 1/2
α = 0.75
α = 1.0
α = 1.5
α = 2.0
α = 2.5
α = 3.0
α = 4.0
α = 5.0
α = 6.0
α = 8
α = 10
VALUES OF $\phi_D = \frac{P_D}{H} \times 100$
50
40
30
20
10
0
0.9
1.0


SHEET PILE NOT AT END
W.L.
H
b
E C
d
D
α = b/d
$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$
$\phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$
$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$ - „ -
TO FIND ϕ_E FOR ANY VALUE OF α AND BASE RATIO $\frac{b_1}{b}$,
READ ϕ_C FOR BASE RATIO $(1 - \frac{b_1}{b})$ FOR THAT VALUE OF α
AND SUBTRACT FROM 100
THUS ϕ_E FOR $\frac{b_1}{b}$ = 0.4 AND α = 4, = 100 − ϕ_C FOR $\frac{b_1}{b}$ = 0.6
AND α = 4, = 100 − 29.1 = 70.9 %
TO GET ϕ_D FOR VALUES OF $\frac{b_1}{b}$ LESS THAN 0.5
READ ϕ_D FOR BASE RATIO $1 - \frac{b_1}{b}$ AND SUBSTRACT FROM 100
THUS ϕ_D FOR $\frac{b_1}{b}$ = 0.4 AND α = 4
= 100 − ϕ_D FOR $\frac{b_1}{b}$ = 0.6 AND α = 4
= 100 − 44.8
= 55.2
VALUES OF 1/α = d/b
0 0.2 0.4 0.6 0.8 1.0
W.S
H
C
E
d
D
b
B
SHEET PILE AT END
SCALE FOR $\phi = \frac{P}{H} \times 100$
20 30 40 50 60 70 7.4
ϕ_D
ϕ_E
DEPRESSED FLOOR
W.S
H
b
D'
d
$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$ ----
$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$ -·-·-
$\phi_{C_1} = 100 - \phi_E$
$\phi_{D_1} = 100 - \phi_D$
$\phi_{D'} = 100 - \phi_{D'_1}$ (DEPRESSED FLOOR)
$\phi_{D'} = \phi_D - \frac{2}{3}(\phi_E - \phi_D) + \frac{3}{\alpha^2}$ (Eqn 7.70) (DEPRESSED FLOOR)
$\lambda = \frac{\alpha + \sqrt{1 + \alpha^2}}{2}$
α = INFINITY
VALUES OF $\phi_C = \frac{P_C}{H} \times 100$
100 90 80 70 60 50 40 30 20 10
α = 20, 25, 30, 18, 16, 14, 12, 11, 10, 9, 8, 7, 6, 5, 4, 3, 2.5, 2, 1.5, 1, 0.75, 1/2, 1/3, 1/4, 1/5, 1/10, 0
CURVES OF ϕ_C
CURVES OF ϕ_D
α = 0
α = 1/10, α = 1/2, α = 0.75, α = 1.0, α = 1.5, α = 2.0, α = 2.5, α = 3.0, α = 4.0, α = 5.0, α = 6.0, α = 8, α = 10.0, α = 12.0, α = 14, α = 16, α = 18, α = 20
VALUES OF $\phi_D = \frac{P_D}{H} \times 100$
50 40 30 20 10 0
0.9 1.0

From what has been stated above it would be noticed that the determining factors in the design of a weir work, are—

(a) the uplift pressure,

(b) the Exit Gradient.

The uplift pressure is resisted by providing suitable floor thickness and the Exit Gradient by providing suitable depth of pile line at the down-stream end. The depth of down-stream pile line is fixed from yet another consideration, *i.e.*, its bottom should be at or below the level of normal flood scour, because if the bottom of the pile is at a higher level, the sub-soil behind the piles may run out by simple earth pressure irrespective of the Exit Gradient. This can happen both up-stream and down-stream, as the scour holes can occur on either side. On the up-stream side, the flow of water would tend to oppose slipping of earth due to earth pressure while on the down-stream side, the Exit Gradient would help this. Failure at the up-stream end would, therefore, occur due to scour, at low heads when the restraint exercised by the seepage flow is less while on the down-stream side, increased head would cause an increase in the Exit Gradient which would help slipping due to earth pressure. Thus the depth of the up-stream pile would be determined only from the depth of scour, while that of the down-stream pile, it will have to be fixed from the depth of scour and also the Exit Gradient. Under normal conditions, the depth of scour up-stream would be higher than that of the down-stream by the afflux produced by the weir obstruction.

Even though a down-stream pile line makes the weir structure stable, it cannot be made of any indefinite depth. This is because the pressure under the down-stream floor increases as the depth of up-stream pile line increases, and this factor limits the depth of the down-stream pile. The up-stream pile line is relatively more effective to destroy the pressure head and reduces uplift pressures under the weir while the pile line at the down-stream end is more effective to reduce the Exit Gradient. The intermediate pile lines simply act as important secondary lines of defence so that even if the impervious floor is damaged at either the up-stream or the down-stream end by failure of the end piles under abnormal scour, the rest of the floor and the super-structure will be saved from collapse by the intermediate piles.

(ii) **Determination of the depth of scour.**—The depth of scour required for fixing the depth of the down-stream pile line is determined either from Kennedy's formula or Lacy's formula. Both these formulae are based on the assumption that for any flood discharge, there is a minimum stable water-way and thus for any flood conditions, there is always a unique value of discharge per ft. run. According to Kennedy's formula $d = 1.11q^{\cdot 6}$—(I).

which has been evolved from his silt formula $Vo = .84d^{.64}$ where Vo is the critical velocity, d=the critical depth and q=discharge per foot run. According to Lacy's formula

$$Vo = \left(\frac{Qf^2}{3.8}\right)^{\frac{1}{6}} \quad \text{and } R = .7305 \frac{Vo^2}{f} \quad \text{where Vo=critical velocity}$$

R=Hydraulic mean depth.
Q=Maximum flood discharge.
f=silt factor.

This gives $R = .7305 \left(\frac{Q}{3.8f}\right)^{\frac{1}{3}}$ .... (II)

The above formula is also expressed in terms of discharge per foot run. For any given flood discharge Q, the minimum stable water-way Pw (wetted perimeter) is given by—

$$Pw = 2.67\sqrt{Q} \qquad \text{(III)}$$

$$\text{Since} \quad q = \frac{Q}{Pw} = \frac{\sqrt{Q}}{2.67} \qquad \text{(IV)}$$

the above formula can be written as

$$R = .9 \left(\frac{q^2}{f}\right) \qquad \text{(V)}$$

All the equations I, II, III, IV and V have been plotted graphically in Plate No. V for f=1 from which the values of d, e, fw, q could be read off for any given value of Q.

In using the PQ formula in connection with determining the water-way, it has to be remembered that it applies to regime channels whose bed and slope profile must conform to an ellipse. The construction of a weir with a level crest alters the regime conditions and the only formula that can apply in this case will be the one based on a discharge per foot run, *i.e.*, $R = .9\left(\frac{Q^2}{f}\right)^{\frac{1}{3}}$. Similarly, the conditions at the under-sluice would be different from those at the weir site and here also the discharge per foot run would be the determining factor for the depth of scour. In the above formula f can be obtained from Lacy's formula connecting Q, slope S and f.

$$S = \frac{1}{178} \quad \frac{f^{\frac{5}{3}}}{Q^{\frac{1}{6}}} = \frac{1}{2480}\left(\frac{f^5}{q}\right)^{\frac{1}{3}}$$

Having determined the depth for the minimum stable water-way from either Kennedy's or Lacy's formula, the depth of scour could be determined by deducting therefrom the actual depth of the impermeable floor below the H.F.L. The talus both up-stream and down-stream is provided in such a length that after scour, the bed with talus overlaid, assumes a slope of 1 in 3 and extends right upto the scour hole as shown below:—

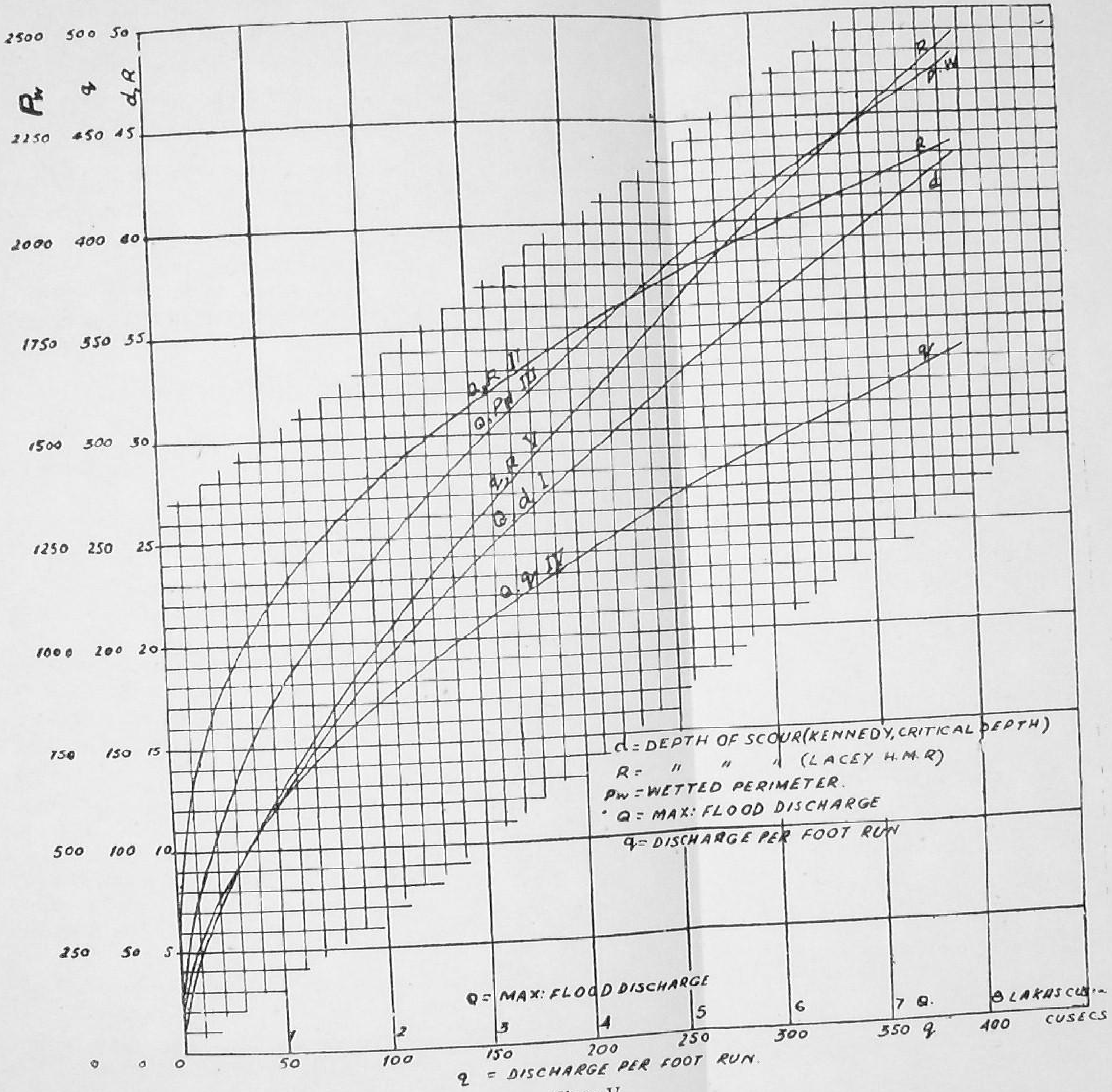

Pw
q
d,R
d = DEPTH OF SCOUR (KENNEDY, CRITICAL DEPTH)
R = " " " (LACEY H.M.R)
Pw = WETTED PERIMETER.
Q = MAX: FLOOD DISCHARGE
q = DISCHARGE PER FOOT RUN
Q,R II
Q,Pw III
d,R V
Q,d I
Q,q IV
R
P.W
R
d
q
Q = MAX: FLOOD DISCHARGE
q = DISCHARGE PER FOOT RUN.
CUSECS

Plate V

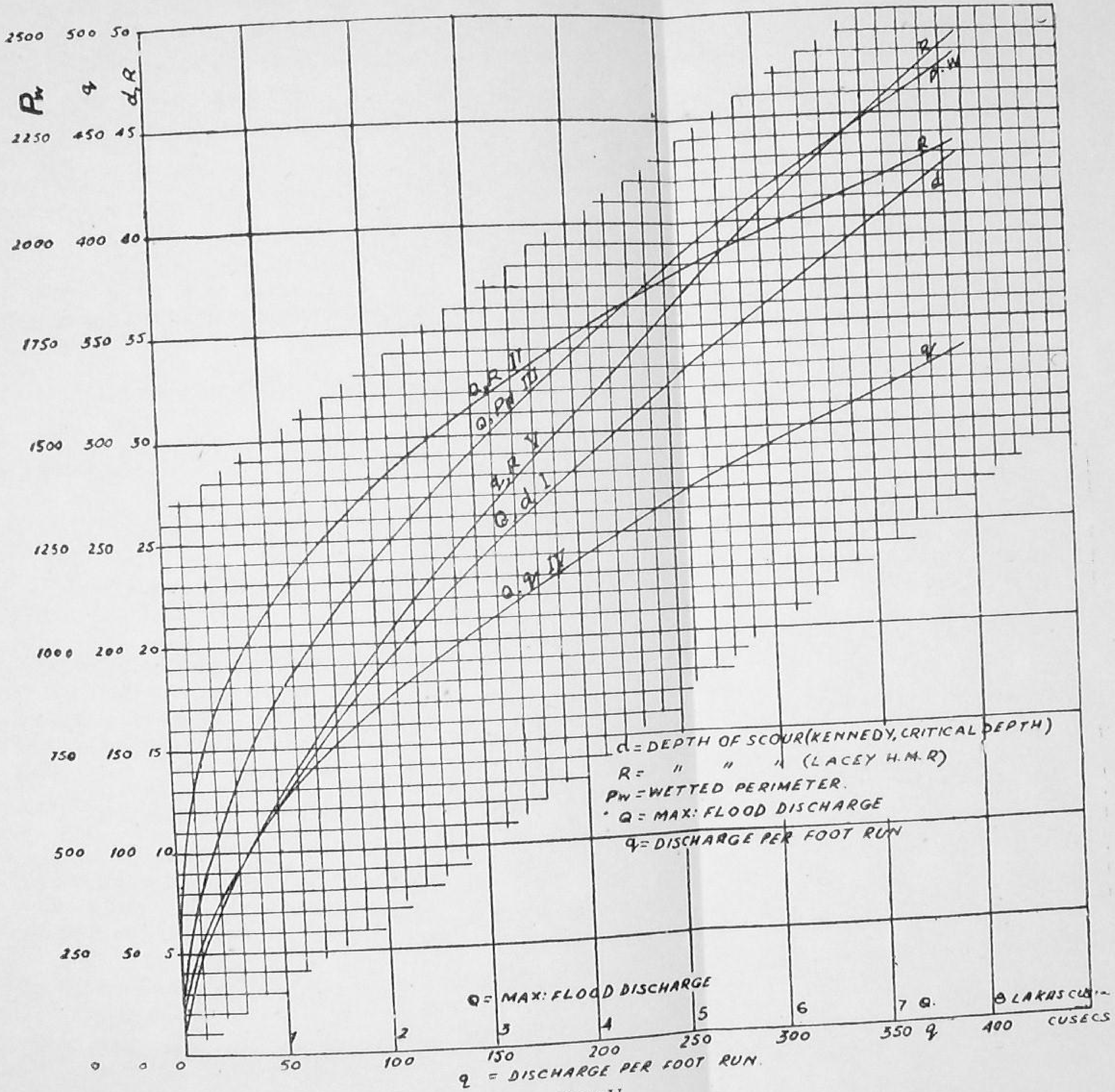
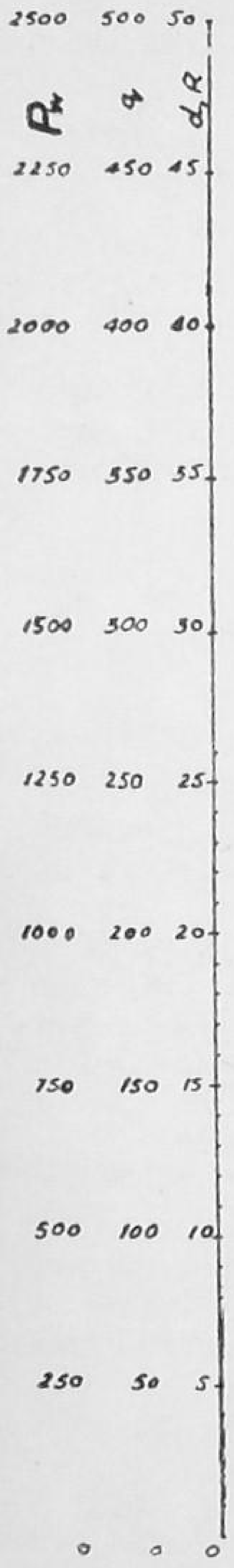

Pw
q
d, R
d = DEPTH OF SCOUR (KENNEDY, CRITICAL DEPTH)
R = " " " (LACEY H.M.R)
Pw = WETTED PERIMETER.
Q = MAX: FLOOD DISCHARGE
q = DISCHARGE PER FOOT RUN
Q = MAX: FLOOD DISCHARGE
8 LAKHS CUSECS
CUSECS
q = DISCHARGE PER FOOT RUN.

Plate V



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Between the down-stream talus and the impermeable floor, an inverted filter is also provided. It consists of a layer of fine bajari or kanker resting directly on the bed soil followed by ballast of increasing grades and finally spawls and boulders. This filter bed is often over-loaded and protected by series of heavy blocks (with open joints) against dislocation by the action of surface flow. The idea is to prevent erosion of the soil by the action of waves and under-mining by seepage flow. This graded filter permits free movement of water but prevents the movement of sand.

(iii) **Example of a Design.**—The process of design can be best explained by an example. Following is the profile of Khanki Weir (Punjab) which affords a good example of how to proceed with a design:—

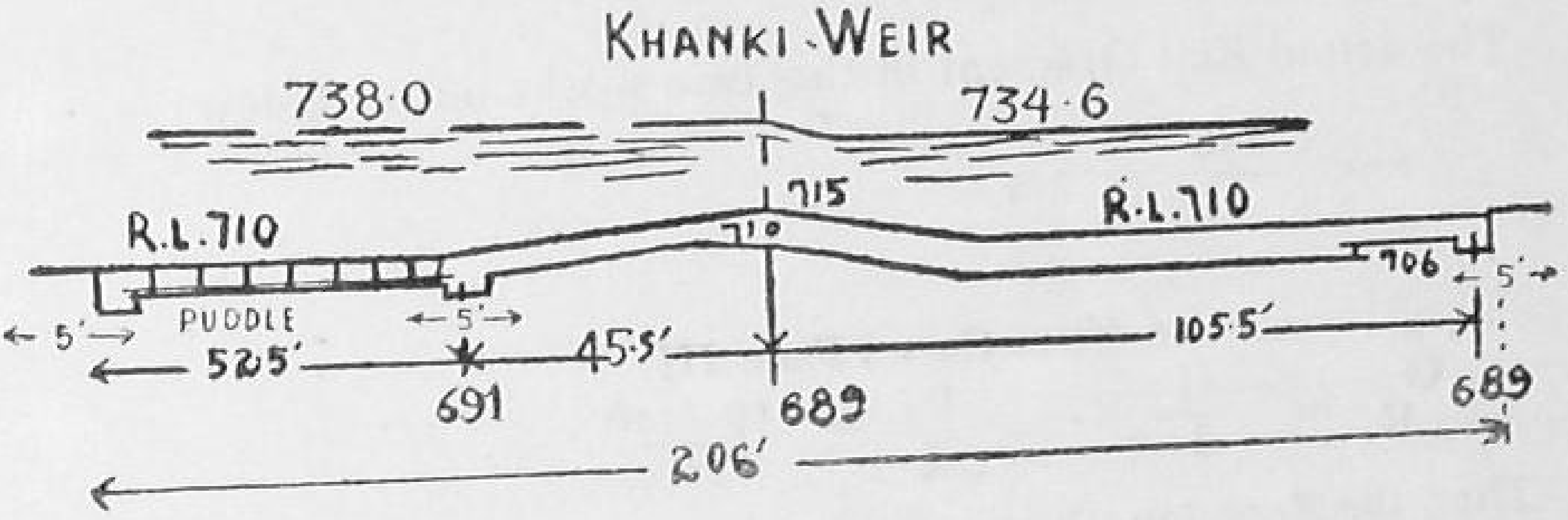

Fig No. 16

H =Total head = 19 ft.
H.F.L.=738.0 up-stream
734.6 down-stream
Afflux=3.4 ft.
q=discharge per foot run = 387 cusecs.

Level of up-stream and down-stream horizontal floor=710·0. This was arrived at from a consideration of the standing wave forming not lower than the toe of the glacis. R=depth of scour=48' (Plate V). Bottom of scour hole down-stream=734·6−48=686·6. Bottom of scour hole up-stream=738—48=690·0. The bottom level of the down-stream piles is actually at 689·0 and of the up-stream piles at 691·0.

d = depth of down-stream pile line below floor
= 710-689 = 21 ft.
d_1 = depth of up-stream pile line below floor
= 710—691 = 19ft. (This does not affect the Exit Gradient)

$$G_E = \text{Exit Gradient} = \frac{H}{\pi d \sqrt{\lambda}}$$

$$\frac{H}{d} = 1/21$$

If factor of safety is 6 so that $G_E = 1/6$

$$\frac{1}{\pi\sqrt{\lambda}} = \frac{21 \times G_E}{19} = \frac{21 \times 1/6}{19}$$

$$= .18$$

From Plate III for a value of $\frac{1}{\wedge\sqrt{\lambda}} = .18$

$$\alpha = 5$$

Thus for a 21 ft. pile line, the floor length to give an Exit Gradient of 1/6 need not have been more than 5×21=105 ft.

The actual floor length=156 ft.

This floor length was put in to give a long enough passage for the standing wave to dissipate its destructive energy before getting on to the previous apron.

The actual Exit Gradient in this case works out as below:—

$$\alpha = \frac{156}{21} = 7.4$$

$$\frac{1}{\wedge\sqrt{\lambda}} = .16 \text{ (from Curve, Plate III)}$$

$$G_E = \frac{H}{d} \times \frac{1}{\wedge\sqrt{\lambda}} = \frac{19 \times .16}{21} = 1/7$$

Thus the floor length is suitable.

To determine the uplift pressures.

Upstream pile line

$b_1 = 156$

$1/\alpha = 19/156 = 0.12$

$\varphi_D = 78.8\%$

$\varphi_C = 69.0\%$

$\varphi_D \ \varphi_C = 9.8\%$

φ_C corrected $= 69.0 + 2.6 + 2.4$

$= 74\%$

ϕ_c correction for depth at R. L. 705

$$= \frac{710-705}{19} \times 9{\cdot}8 = +2.6\%$$

Effect of middle pile on ϕ_c

$$= 19\sqrt{15/45{\cdot}5} \times \frac{16+14}{156}$$

$$= +2.4\%$$

Middle pile line

d=710-689=21 ft

$\alpha = 156/21 \quad 7.4$

$b_1/b = 48/156 = .308$

$\varphi E_1 = 71.2$

$\varphi D_1 = 61.7$

$\varphi C_1 = 5302\%$

No correction for depth is necessary in this case.

Effect of down-stream pile on φ_{E_1}

D=710-691=19'·

d=710-689=21'·

$$C = 19\sqrt{19/45{\cdot}5} \times \frac{(19+21)}{156} = -3.3\%$$

$\varphi E_1 - \varphi D_1 = 9.5\%$

$\varphi D_1 - \varphi C_1 = 8.5\%$

φE_1 corrected $= 71.2 - 3.3$
$= 67.9\%$

φC_1 corrected $= 53.2 + 2.2$
$= 55.4\%$

Downstream pile line

$d = 710 - 689 = 21$

$\propto = 21/156 = .134$

$\varphi E_2 = 32.5\%$

$\varphi D_2 = 22.2\%$

$\varphi E_2 - \varphi D_2 = 10.3\%$

ϕE_2 corrected $= 32.5 - 2 - 1.5$
$= 29.0\%$

Effect of down stream pile on φC_1

$D = 710 - 689 = 21'$.

$d = 710 - 689 = 21'$.

$$C = 19\sqrt{21/105.5} \times \frac{21+21}{156} = +2\ 2\%$$

correction for depth $= 4/21 \times 10.3$
$= -2\%$

Effect of middle pile line

$D = 17'$, $d = 17'$

$$C = 19\sqrt{17/105.5} \times \frac{17+17}{156} = -1.5\%$$

Comparative Table

	U.S. Pile		Middle Pile			D.S. Pile	
	ϕD	ϕC	ϕE_1	ϕD_1	ϕC_1	ϕE_2	ϕD_2
Calculated	78.8	74.0	67.9	61.7	55.4	29.0	22.2
Experimental	81.6	73.0	67.9	61.3	53.7	29.6	22.2
Difference	2.8	1.0	0.0	0.4	1.7	0.6	0.0

For a head of 19′ the absolute uplift pressures work out as below:—

$$P_C = \frac{19 \times 74}{100} = 14.1$$

$$P_{E_1} = \frac{19 \times 67.9}{100} = 12.9'$$

$$P_{C_1} = \frac{19 \times 55.4}{100} = 10.6'$$

$$E = \frac{19 \times 29}{100} = 5\ 5'$$

The floor thickness below the middle pile line can now be calculated, taking the submerged weight of stone masonry or concrete as 1·4 times that of water. The pressures along the floor between the middle and down-stream pile lines can be obtained by assuming a linear drop from 10·6 to 5·5 ft. along the length between the two pile lines.

8. **Weir Shutters.**—There are various types of crest shutters used in different parts of the country. Here only two common types would be described:--

(i) **Ashford's Patent Drop Shutters.**—As shown in Fig. 17, the shutters consist essentially of a flat steel framed hinged gate supported on the down-stream side by an inclined hinged strut which is free at its upper end to slide in a groove fitted to the back of the shutter. When the shutters are up, this strut is held in position by a locking bar and trip lever. This trip lever (which is really a hinged extension of the locking bar) projects two or three inches beyond the face of its shutter when in its closed position and thus overlaps the adjacent shutter by a similar amount. When any shutter falls, it automatically knocks forward the trip lever projecting from the adjacent shutters which allows the locking bar thereof, to drop into a vertical position, thus leaving the supporting strut free to slide upwards in groove. These shutters are always either in a fully raised or fully open position. The shutters are usually not constructed more than 6 ft. in height. The usual method employed for raising the shutters is by means of winch and tackle mounted on a suitable boat. To start with, the boat is moored to one of the abutments and having lifted the first two or three shutters from this position, it is then secured to the up-stream face of the shutters from where it proceeds along the line lifting the shutters in advance. This system is slow and constitutes one of the main drawbacks to the use of shutters of this type. It is, however, reasonably effective and has the great merit of cheapness. Mechanical scoops and travelling gantries have been tried as alternatives but have not resulted in large improvements. To lower or drop the shutters, all that is necessary is to release the trip lever of the first shutter in a line by means of a special hand operated gear provided for the purpose after which the rest of the shutters in that line would drop automatically one after the other, the action of dropping any shutters automatically realising the locking lever of the adjacent shutter and so on. Ashford's drop shutters have been used extensively on the weirs in Nothern India, but so far these have not been installed in the South.

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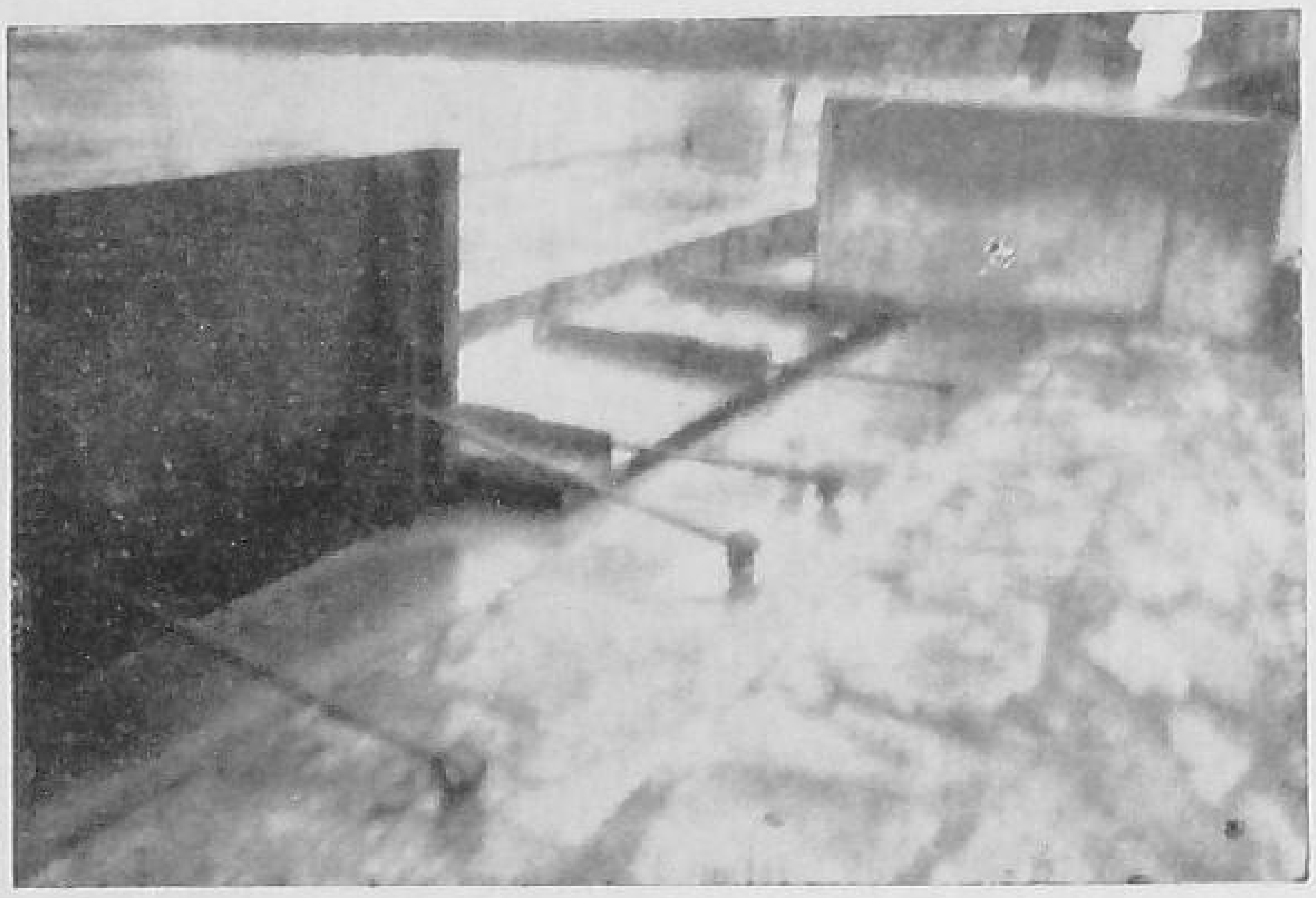
*Extract from "A note on the use of Automatic Falling Shutters on Weirs in India" by W.G. Wheatley.



The Collapsible Shutters on the crest of the Madhaborida Anicut in Orissa

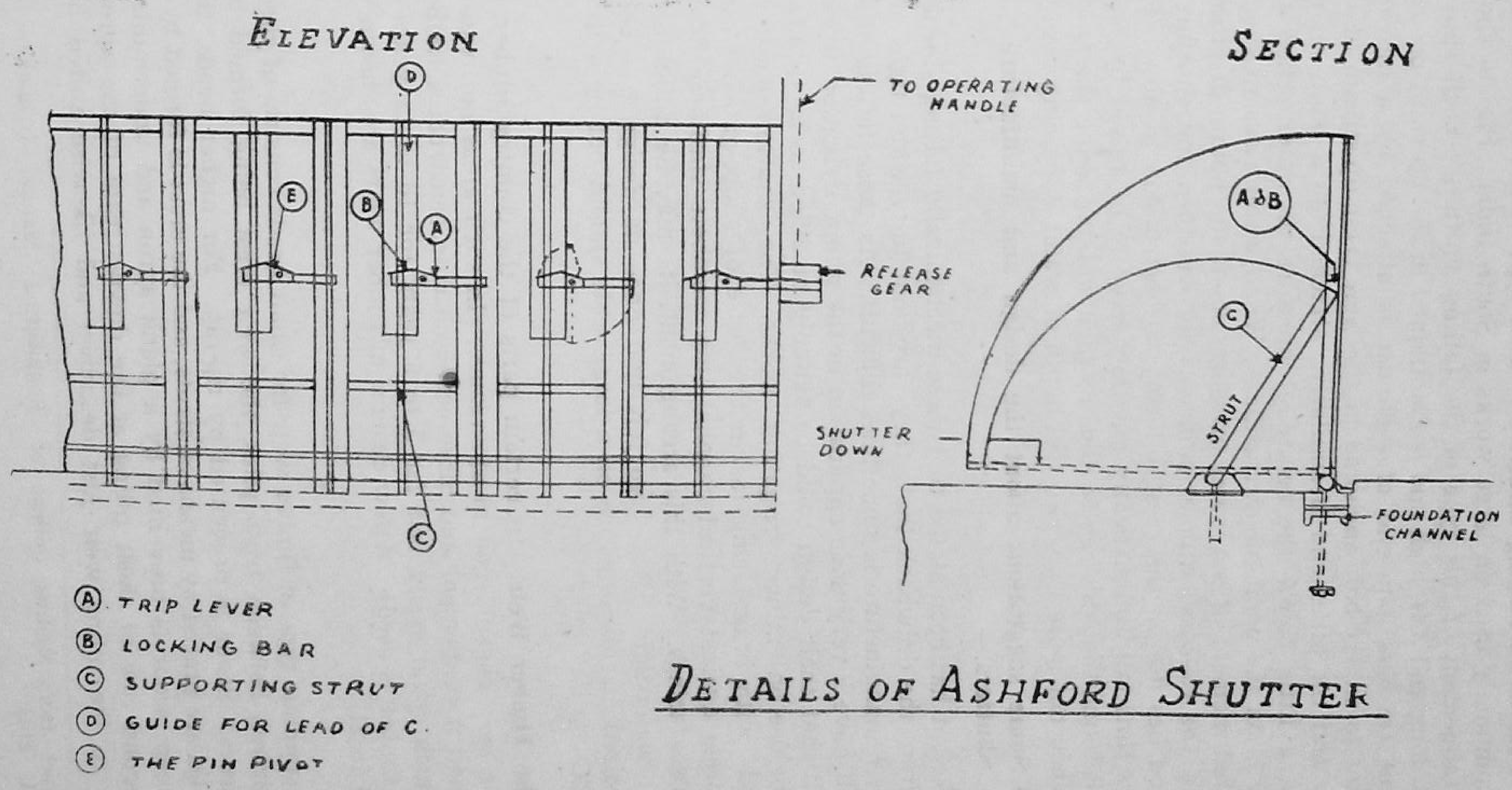

ELEVATION
SECTION
TO OPERATING HANDLE
RELEASE GEAR
SHUTTER DOWN
A&B
STRUT
FOUNDATION CHANNEL
A TRIP LEVER
B LOCKING BAR
C SUPPORTING STRUT
D GUIDE FOR LEAD OF C.
E THE PIN PIVOT
DETAILS OF ASHFORD SHUTTER

Fig 17

(ii) **Fouracre's Falling Shutters***.—This type of shutter has been most commonly used on large works in South India. Fig. 18 shows the arrangement of this type of the falling shutters. Each shutter is 10 ft. long and 3 ft. high and is maintained in the up right position by three tie bars, one end of each bar is attached by a movable pin joint to an eye-bolt anchored in the weir crest up-stream of the shutter and the other end is similarly attached to a point on the shutter, a little below the centre of pressure of water when the shutter is on the point of overturning. When water rises to such a level that moment of the water pressure, about the points of attachment of the tie-bars becomes sufficient to over-turn the shutter, it falls and lies flat on the crest, offering no obstruction to flow. The shutters thus fall automatically but they have to be raised by hand and it is found that this can be done by men walking along the anicut crest when not more than two feet depth of water is passing over it.

(iii) **Some suggestions about the design and manufacture of Falling Shutters:—**

Earlier, it has been stated that these mechanically falling shutters suffer from the drawback that they are not fully water-tight and that their manufacture is sometimes difficult. As these falling shutters will have a very wide application in the minor irrigation works, it is felt that their design could be suitably modified so as to adapt them for use on minor irrigation works. The shutters could be provided with brass-facing (as in the case of sluice-gates) which could press against vertical channel iron members similarly fitted with brass-facings. With this arrangement, it may be possible to make it water-tight. It is also felt that once its design has been standardized, its manufacture on a large scale should not be a difficult task.

9. **The Timber Weir**.—In certain parts of the country timber is available very cheaply and in abundance. In such areas cultivators would find it convenient and more economical to construct a timber weir which could easily last for 5 to 6 years (or 10 to 12 years if treated timber is used). A few designs of timber weirs are shown in Figs. 19 to 21.

The simplest type of timber weir for small heads consists of sheet piling driven between truss piles, the weir crest being formed by wale pieces. Scour is prevented by rip-rap. For higher heads, the truss piles are braced by means of struts, and the crest is formed by a timber cap; scour is prevented by a plank apron and undermining by a second row of sheet piling at the down-stream of the apron. Similar to the piling weir just described and likewise suited for

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*Extract from Madras College of Engineering Manual—Irrigation—by W. M. Ellis.

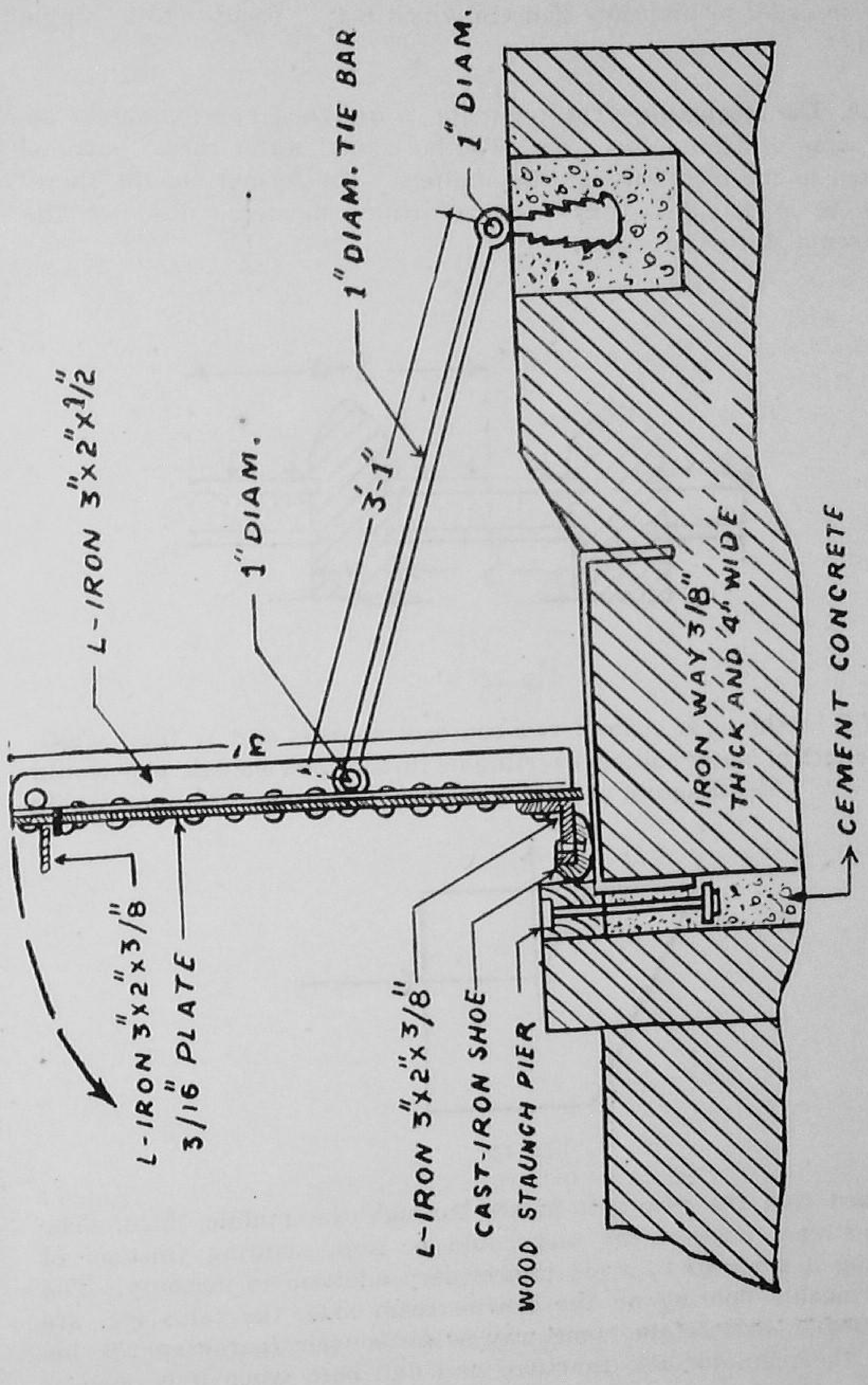

L-IRON 3"x2"x1/2
1" DIAM.
3'-1"
1" DIAM. TIE BAR
1" DIAM.
1 m
L-IRON 3"x2"x3/8"
3/16" PLATE
L-IRON 3"x2"x3/8"
CAST-IRON SHOE
WOOD STAUNCH PIER
IRON WAY 3/8" THICK AND 4" WIDE
CEMENT CONCRETE

Fig 18

higher heads is the so-called 'timber wall weir'. With this type, the river bed is protected by stone paving divided into square fields by sills in order to limit any damage which may occur to a single field.**

10. **The Regulator.**—The regulator is designed approximately on the same principles as a weir. The horizontal water thrust is transmitted to the piers through the shutters. The former should, therefore, be made safe against the overturning moment due to the horizontal thrust of water.

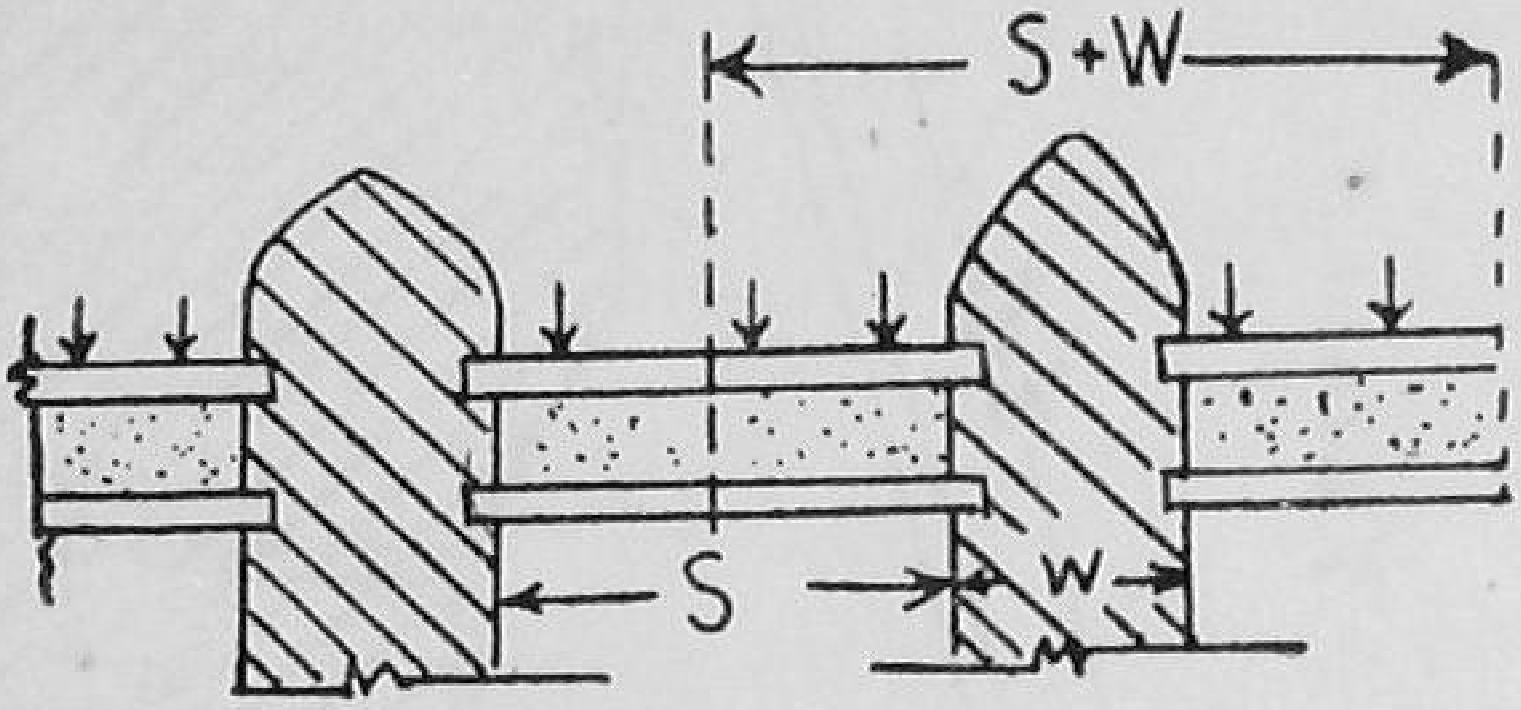


Fig. 22

If S be the clear span of two consecutive piers and W their width the length of water column exerting its thrust upon a single pier would be S+W. The diagram of forces is then drawn and the pier profile

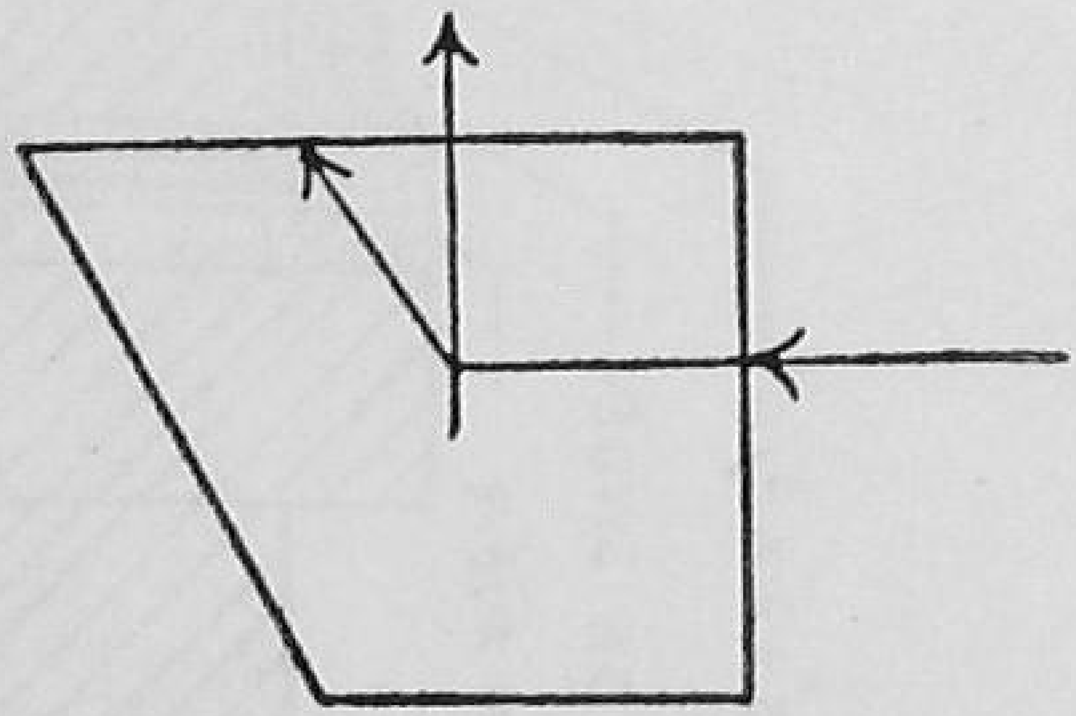


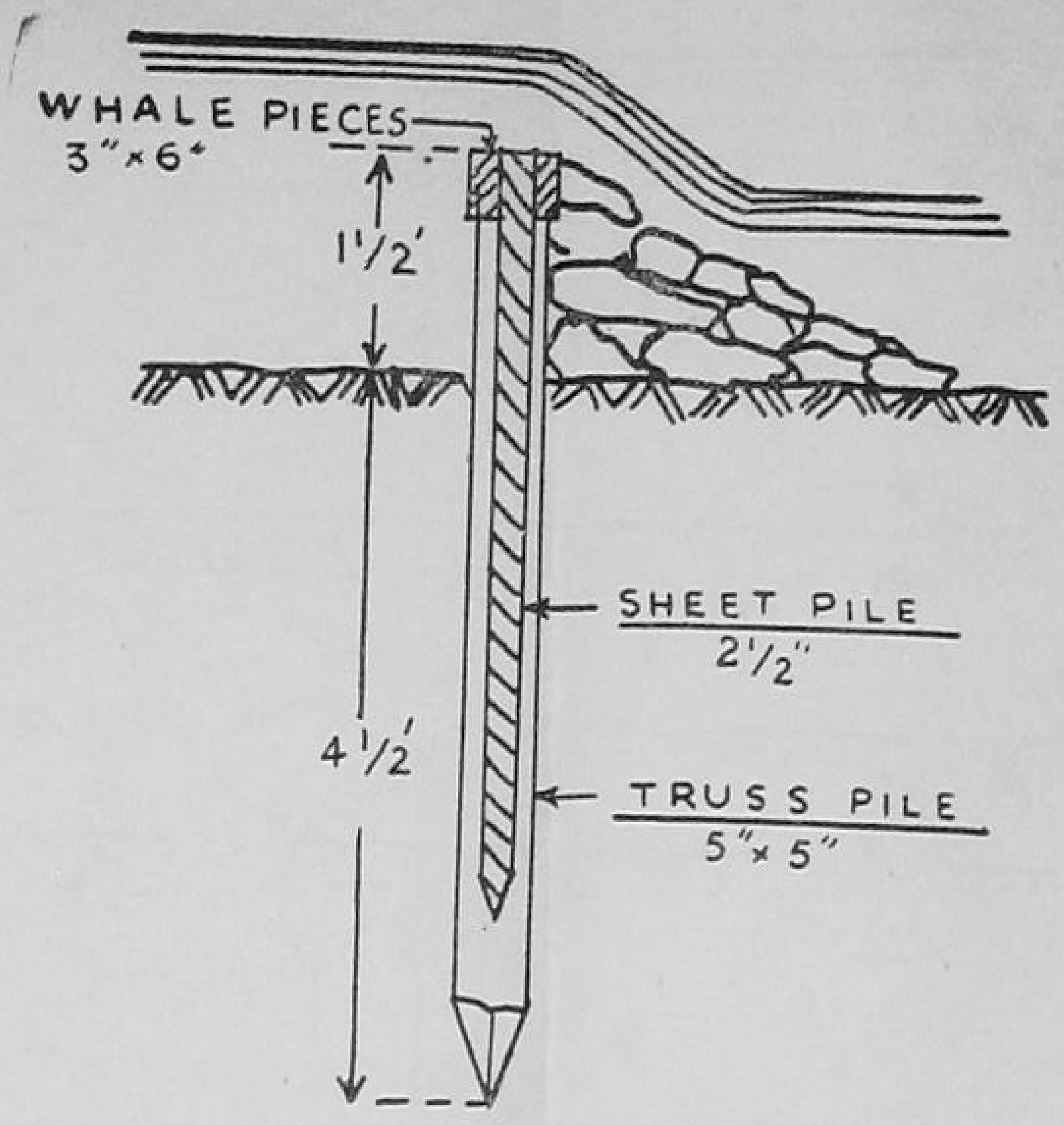
Fig. 23

so fixed that the resultant passes through the middle third. The down-stream nose of the pier could be kept slanting (instead of keeping it vertical) to avoid unnecessary addition to masonry. The impermeable flooring on the down-stream side, the talus etc. are designed in exactly the same way as for a weir (paragraph 3), because the regulator also functions as a fall both when it is open or

**Hydraulic Structures—by A. Schoklitsch pp. 569—570.

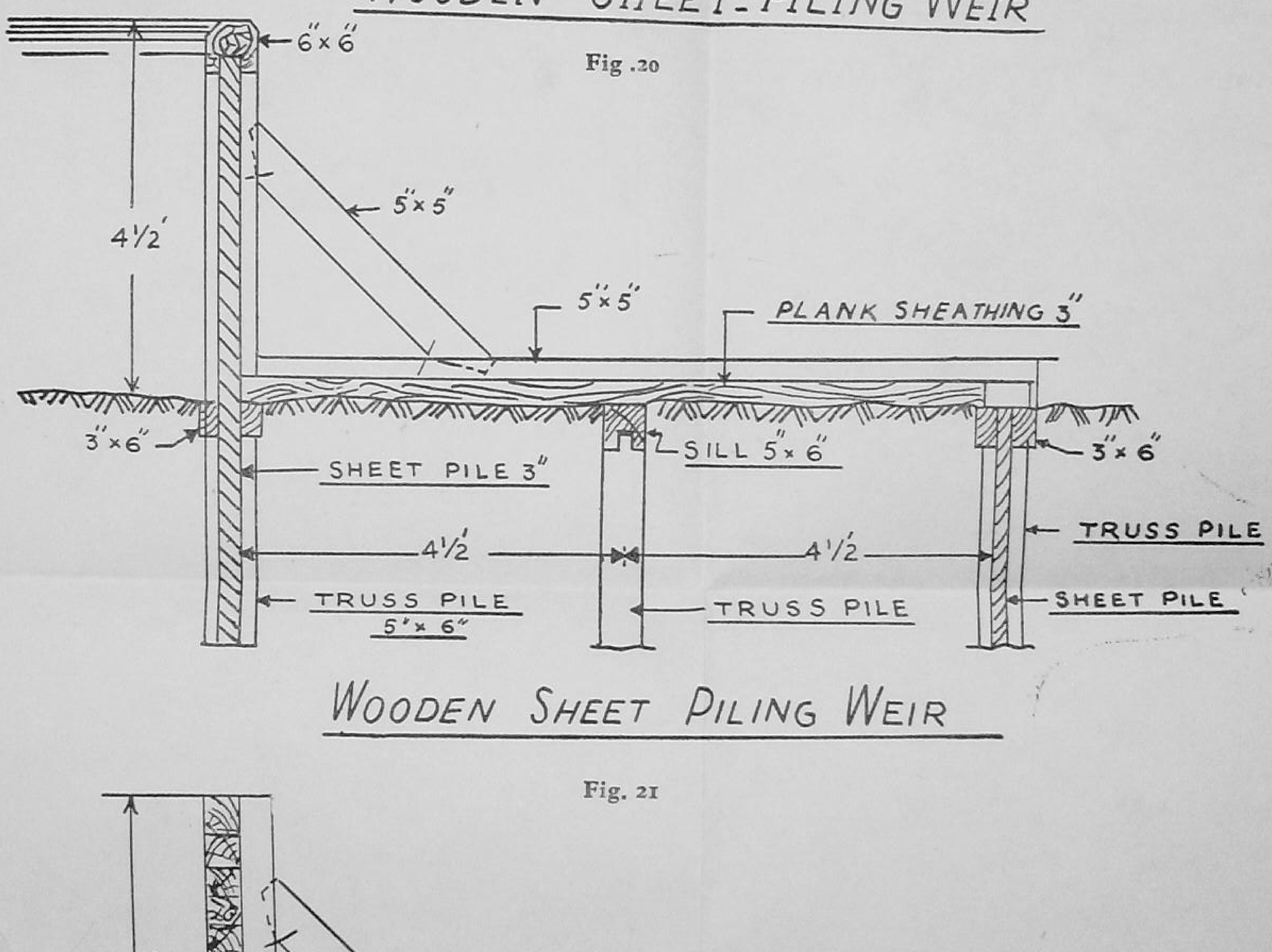
does

Fig. 19



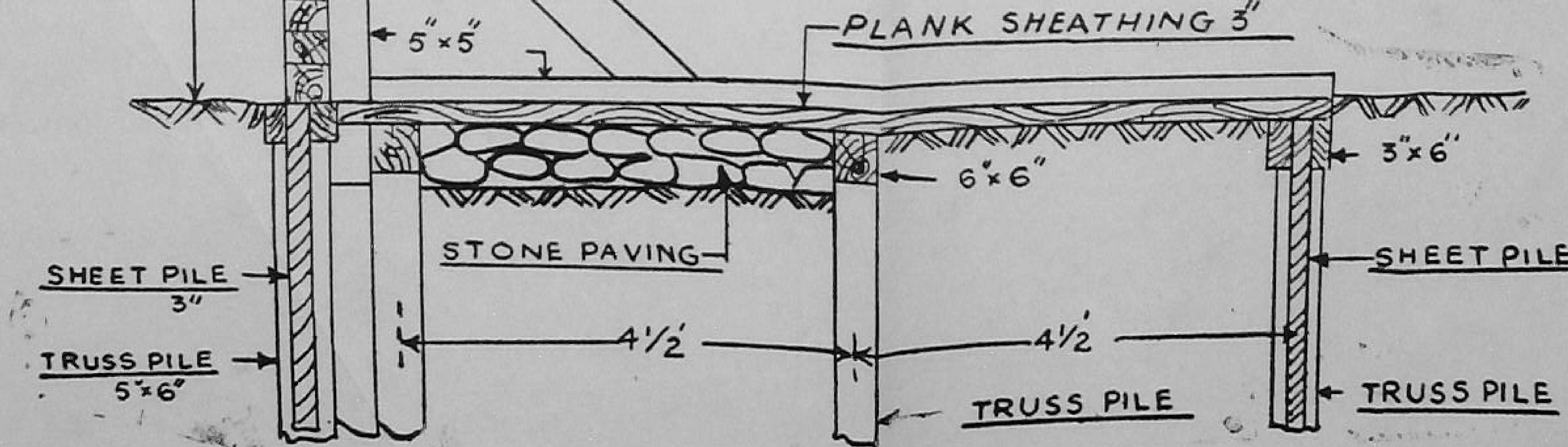
WOODEN SHEET-PILING WEIR

Fig. 20



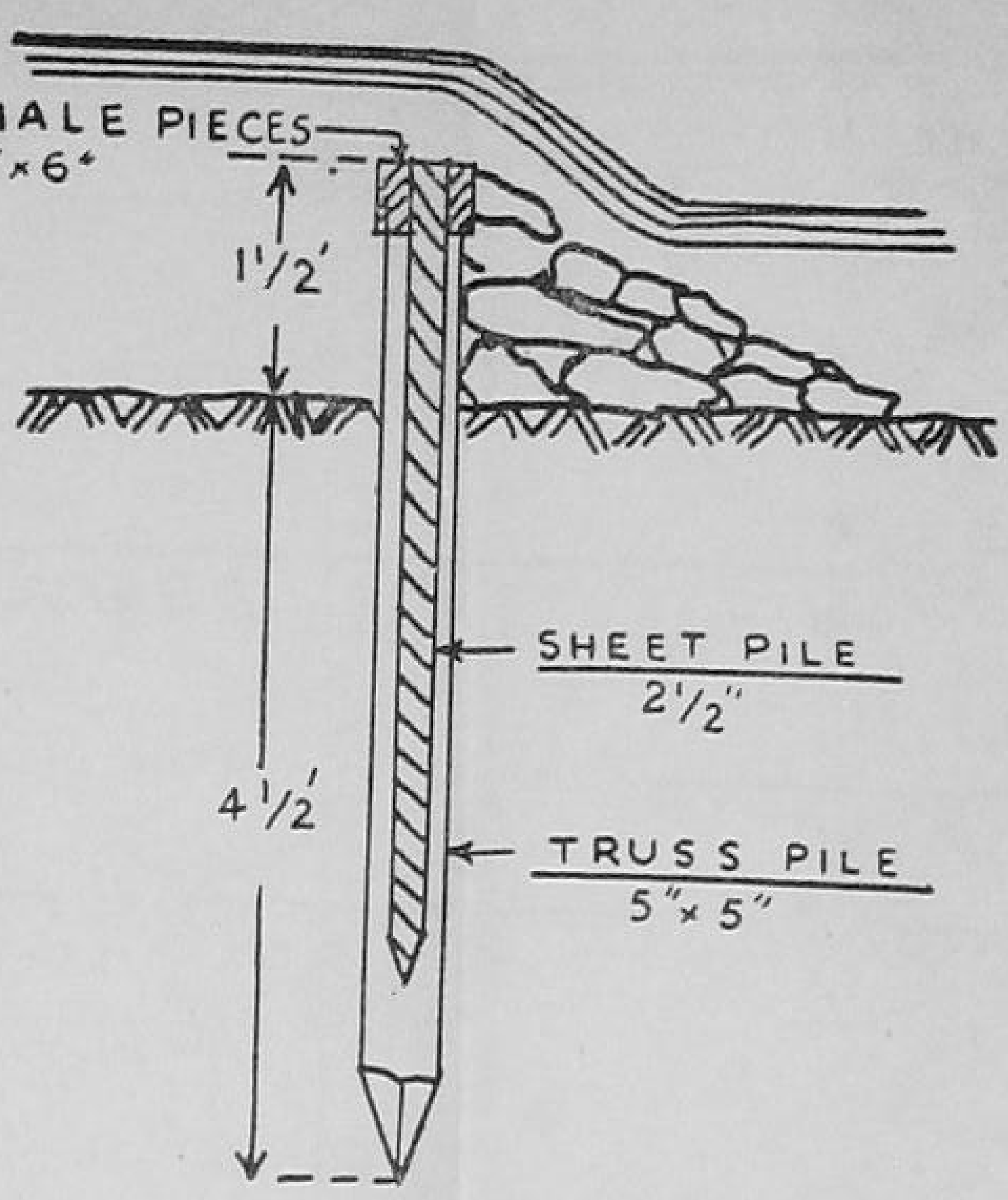
WOODEN SHEET PILING WEIR

Fig. 21



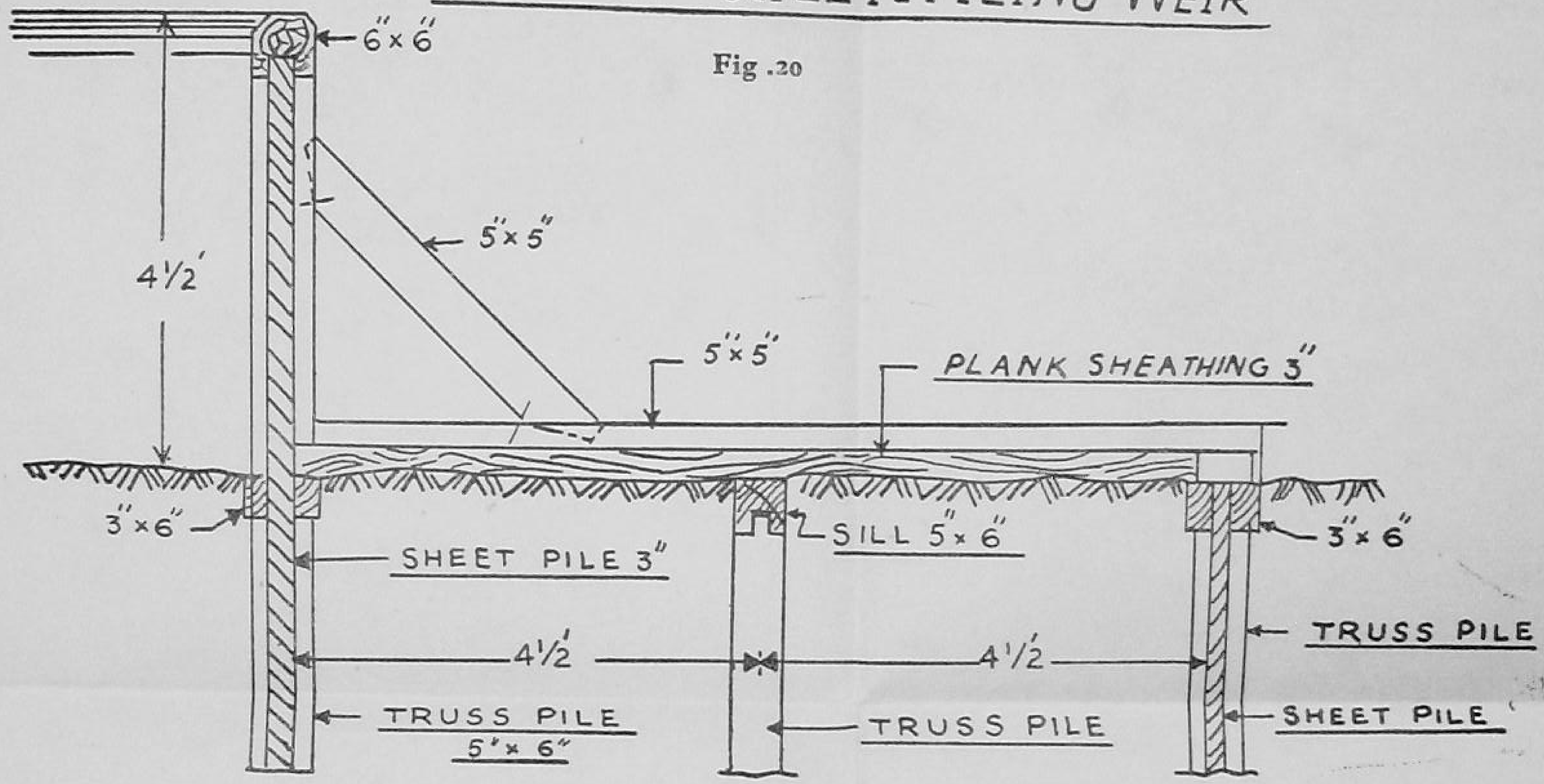
TIMBER WALL WEIR

Fig. 19



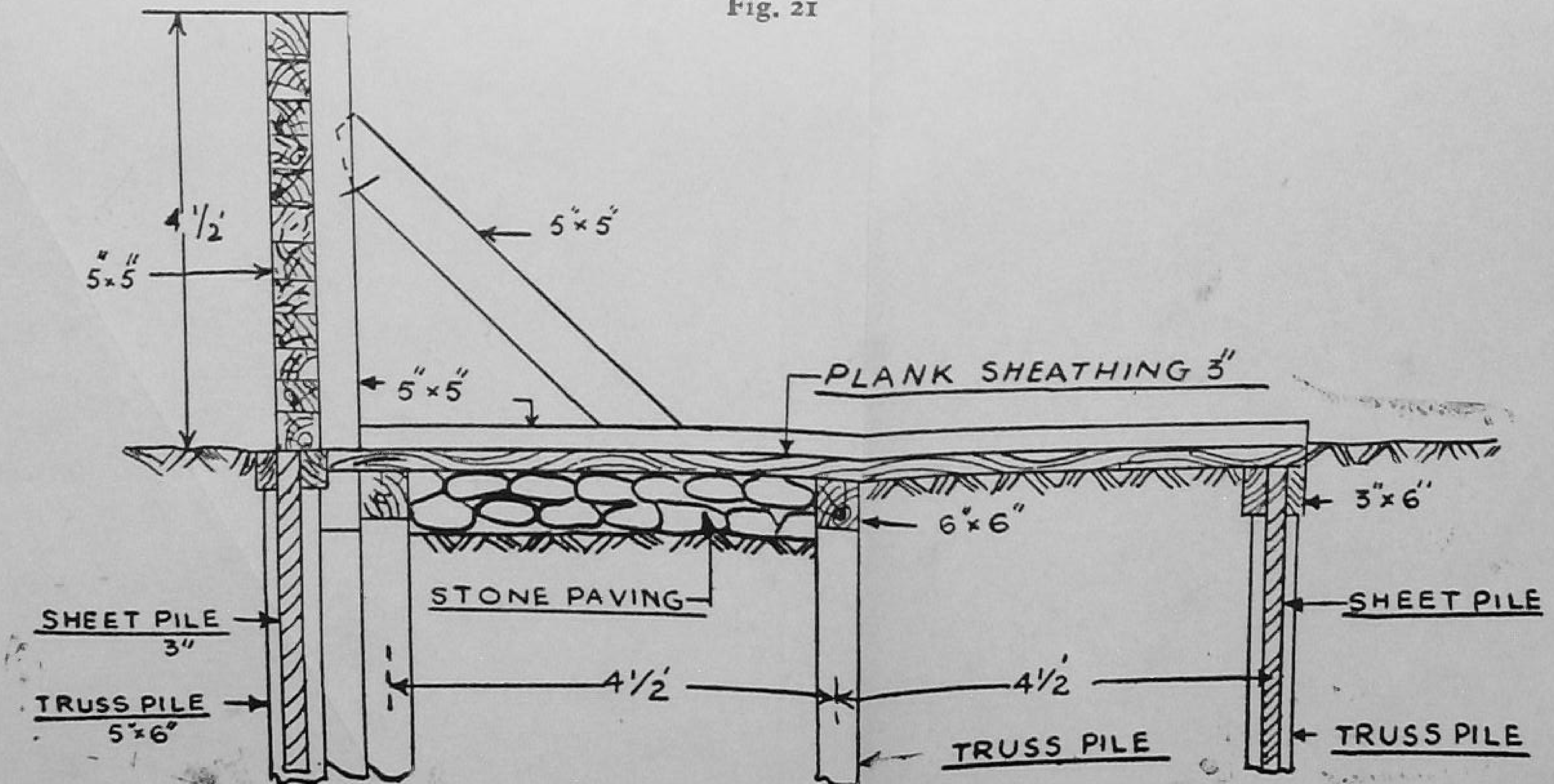
WOODEN SHEET-PILING WEIR

Fig. 20



WOODEN SHEET PILING WEIR

Fig. 21



TIMBER WALL WEIR

esigned in exactly the same way as for a well (paragraph 3), be-
ause the regulator also functions as a fall both when it is open or

**Hydraulic Structures—by A. Schoklitsch pp. 569—570.

The Sihora Regulator in Jabalpur District (Madhya Pradesh). It does Flood Irrigation in 5000 acres every year.

when it is closed. The wooden sleepers are sometimes fitted with iron rings for facility of operation *vide* sketch shown below:—

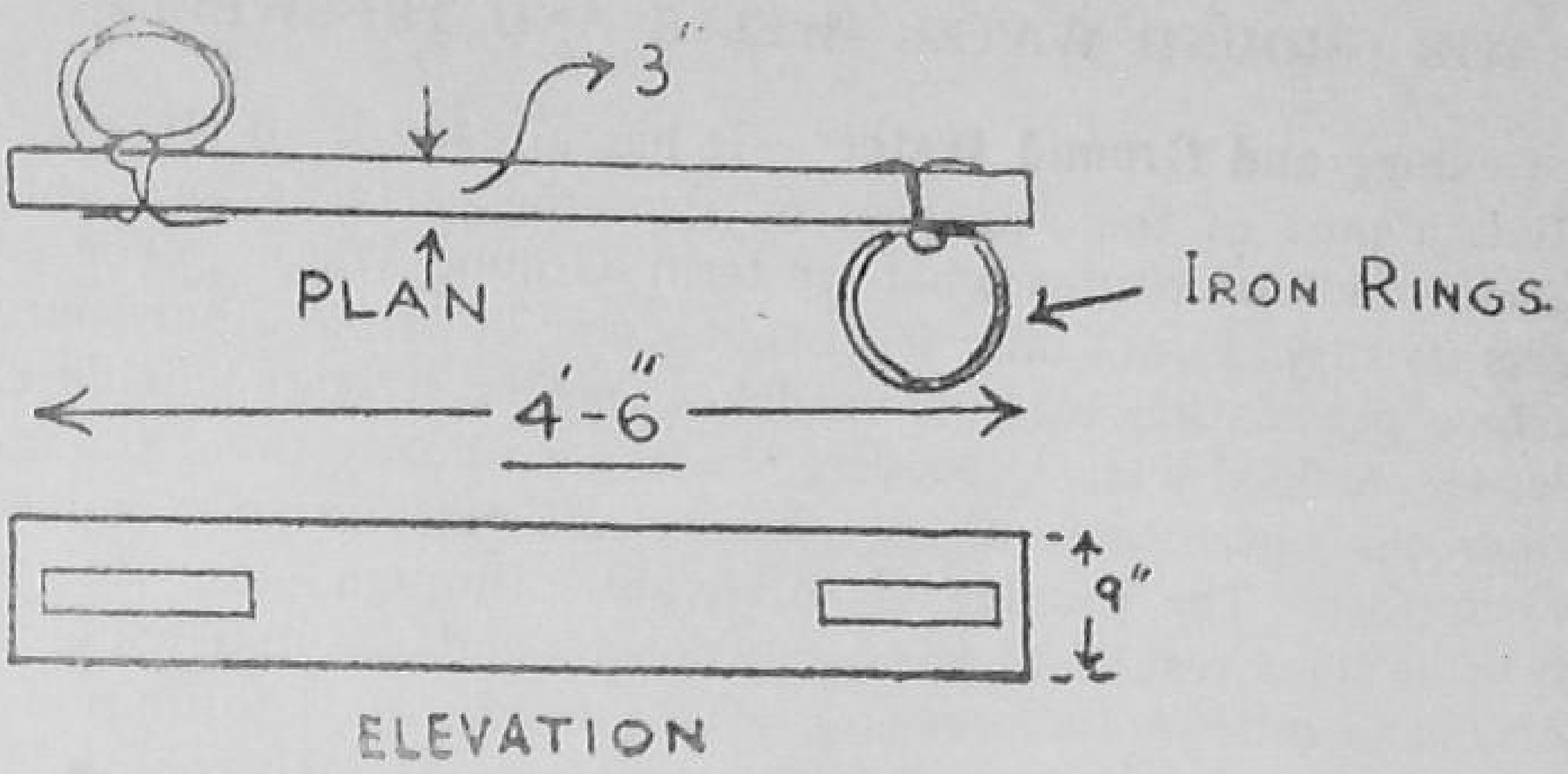


Fig. 24

With these rings fitted it is possible to remove the sleepers even with 3 to 4 feet of water passing over it through the regulator openings. Two men each holding an iron bar about 6' to 8' long ending in a hook stand on opposite piers and manage to engage the hook into the two rings attached to the sleeper. Once the hook is engaged the sleeper can be lifted without much difficulty. A common variation of the horizontal "Karri shutters" is the use of needles, *i.e.*, vertical wooden planks abutting against horizontal members permanently embedded in the piers at the top, and floor projection at the bottom.

11. **Flood Irrigation.**—In some parts of the country, it is a common practice to divert the flood water of a stream into bunded fields by the construction of suitable diversion works. The idea is two-fold—one is that the flood waters (specially the early floods) bring with them large quantities of silt which acts as a manure. When this water is trapped in the bunded fields, the silt is deposited which proves very beneficial to the crop. The other advantage is that by keeping the bunded fields flooded with water throughout the rains, the winter crop can be raised without irrigation, as the soil is fully charged with moisture before the crop is sown. The effect is the same as that of the sub-merging tanks (see Chapter V) which impound the rain water from their own catchments. By diverting the flood water in the above manner, it is sometimes possible to do artificial flooding of large areas of bunded fields which have no catchments of their own. This practice is very commonly followed in the Haveli areas of the Jabalpur and Hoshangabad districts in Madhya Pradesh.

CHAPTER VII

THE GROUND WATER—WELLS AND TUBEWELLS

1. **Geology and Ground Water***.—It has already been stated earlier that a part of the rain that falls on the earth surface, filters through it and constitutes what we term 'Ground Water'. The subsoil gets its supplies not only from rain water but from other sources of surface percolation *viz.*, rivers, lakes, canals, irrigated fields etc. Besides these, flood waters of rivers which periodically cover the land on either side, percolates in the sub-soil and replenish the sub-soil water supplies. The passage of this water through the sub-soil which sometimes results in the formation of underground water reservoirs, is so intimately connected with the geological formation of the tract that a knowledge of this would be extremely helpful in tackling problems connected with ground water supplies. The strata most commonly met with can be divided into two classes—

(i) Pervious formation which is water bearing like sand and gravel.

(ii) Impervious formation which is not water bearing like pure clay, rock and unfissured trap.

The simplest case of sub-soil flow is represented below:

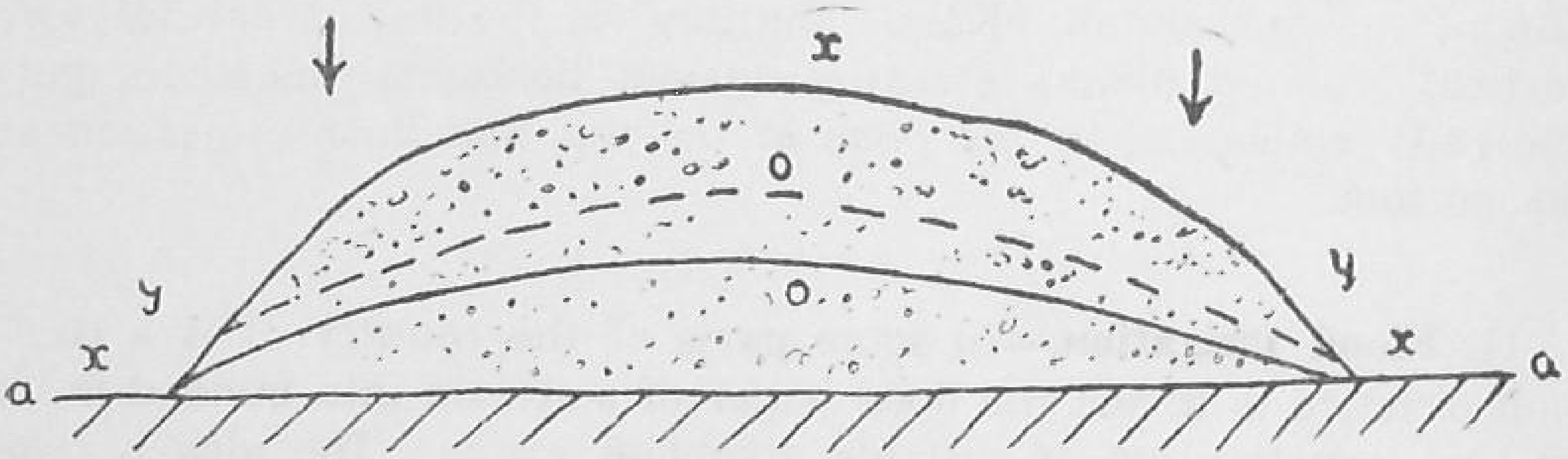


Fig. 25

aa is the surface of some impervious rock underlying a porous layer of sand and gravel. Water falling on the surface xx'x during the wet weather penetrates and sinks downwards till it strikes the impervious base. At the junction of permeable and impermeable strata *i.e.*, at x, water tries to escape and gives rise to springs. The line of saturation which also gives the slope of the ground water plane, could roughly be indicated by the dotted line xox in the above sketch. During the rainy season, the ground water plane has a tendency to rise and the springs will occur higher up *i.e.*, at y. On the other hand, in the dry weather, the plane will fall lower and lower and the spring will have a tendency to diminish or even disappear if the gradient is not sufficient to produce the flow.

*Hydrology and Ground Water by J.M. Lacey, pp. 40—45.

The above is a typical example showing how certain geological configuration of the sub-soil favours the formation of springs. Another condition commonly met with is the occurrence of depression in the underlying impervious stratum as shown below:

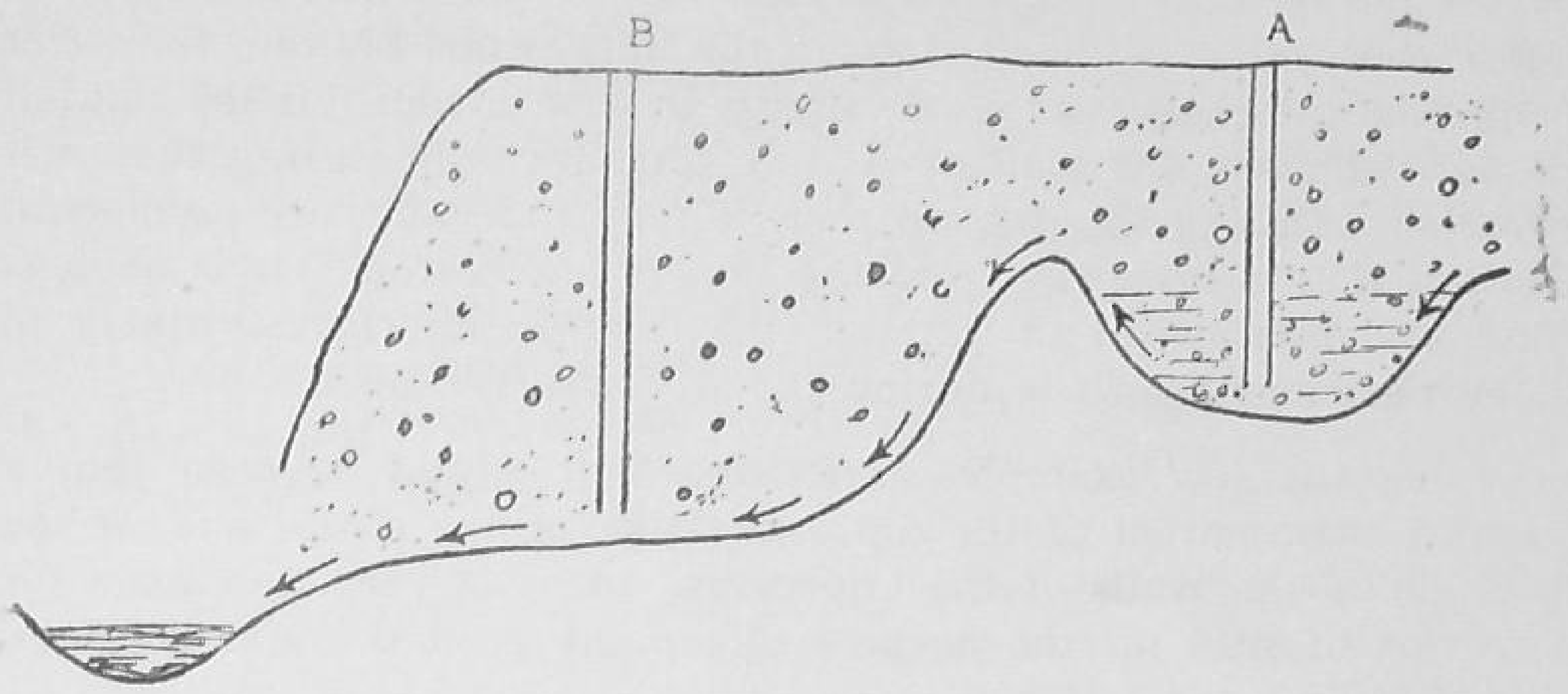


Fig. 26

The depression in the impervious rock, if it is free from fissures, may give rise to an underground reservoir so that the well bored at 'A' into this reservoir becomes a permanent source of water supply. In such a well, water may be available at a much shallower depth, depending upon the position of fold in the rock, than at another well 'B' where water may be available only at considerable depths. The water level in 'B' would depend upon the line of saturation and if, due to presence of a valley, short distance away, the slope of the ground water plane is very steep, it may not be possible to catch any water in the well at 'B', as the entire water will be escaping away into the valley. In such a case, there will, obviously be no permanent spring to feed the well.

There is yet another variation of the geological formation which is also fairly common. This is shown below:—

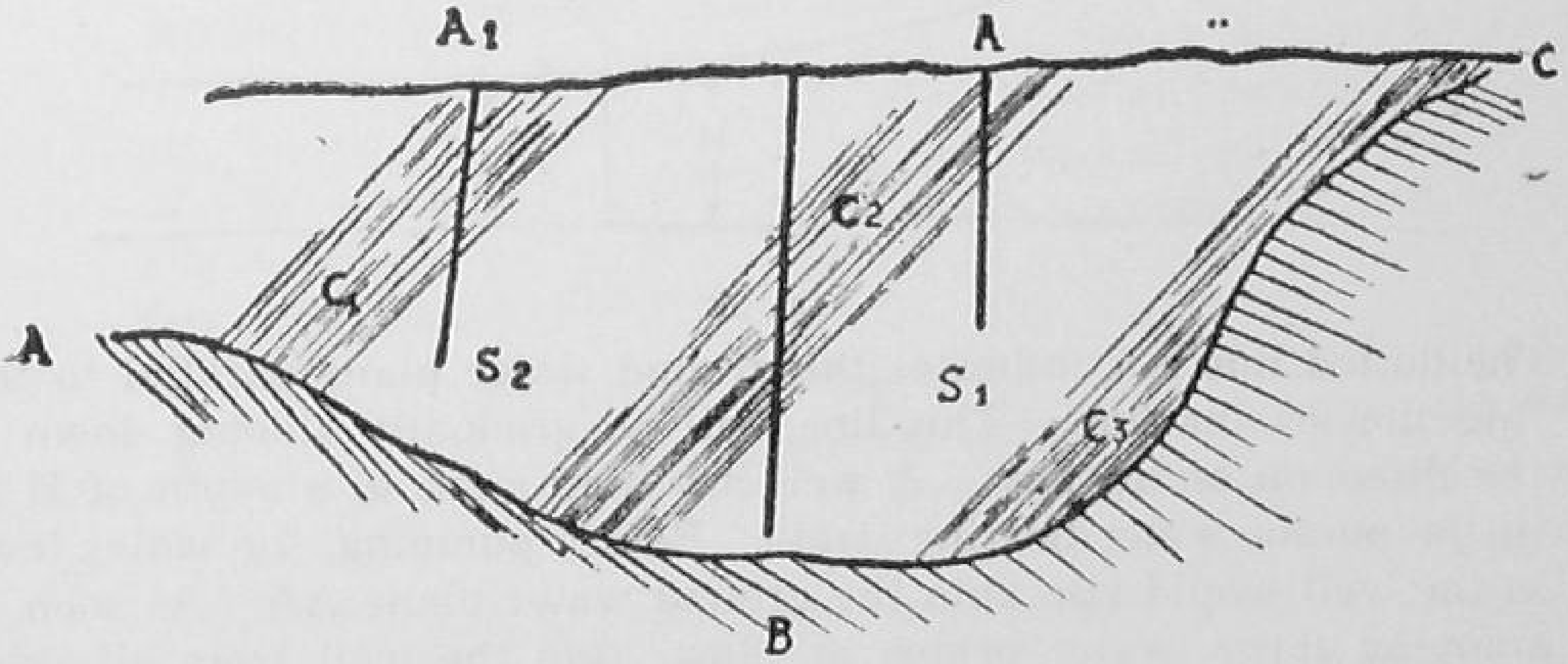


Fig. 27

ABC is the impervious base. C_1, C_2 and C^3 are layers of clay which because of their impermeability, serve as underground dams for the water stored in porous strata S and S_2 The actual yield from wells bored at A and A would depend upon the quantity of water falling on the exposed surface of the permeable strata. The more this well is situated close to the impervious bottom, the surer would be the chances of water supply in such a well. If the rainfall is not sufficient, the well at A may function only during the wet weather. During the dry weather, it may not have any permanent springs if it is far away from the impervious bottom. It is obvious that a well bored at 'B' through the intervening clays is likely to have permanent springs during the dry weather also.

From the few examples given above, it would be seen that a careful examination of the dip and strike of the strata and of the position of the walls of the impervious rocks is very necessary for selection of sites for the purpose of tapping ground water supplies.

2. **Flow of Ground Water into a Well—the Safe Yield.**—The simplest method of tapping the sub-soil water is through the open percolation well. It is a common occurrence that when water is pumped from a well, the water level goes down and then becomes stationary. This lowering down of the water level is known as the "draw down" or "depression". The extent to which the water level falls is known as the "depression head" and this depends upon the rate of pumping *i.e.*, the discharge. What actually happens when water is pumped from a well, could best be analysed with the help of the following sketch:—

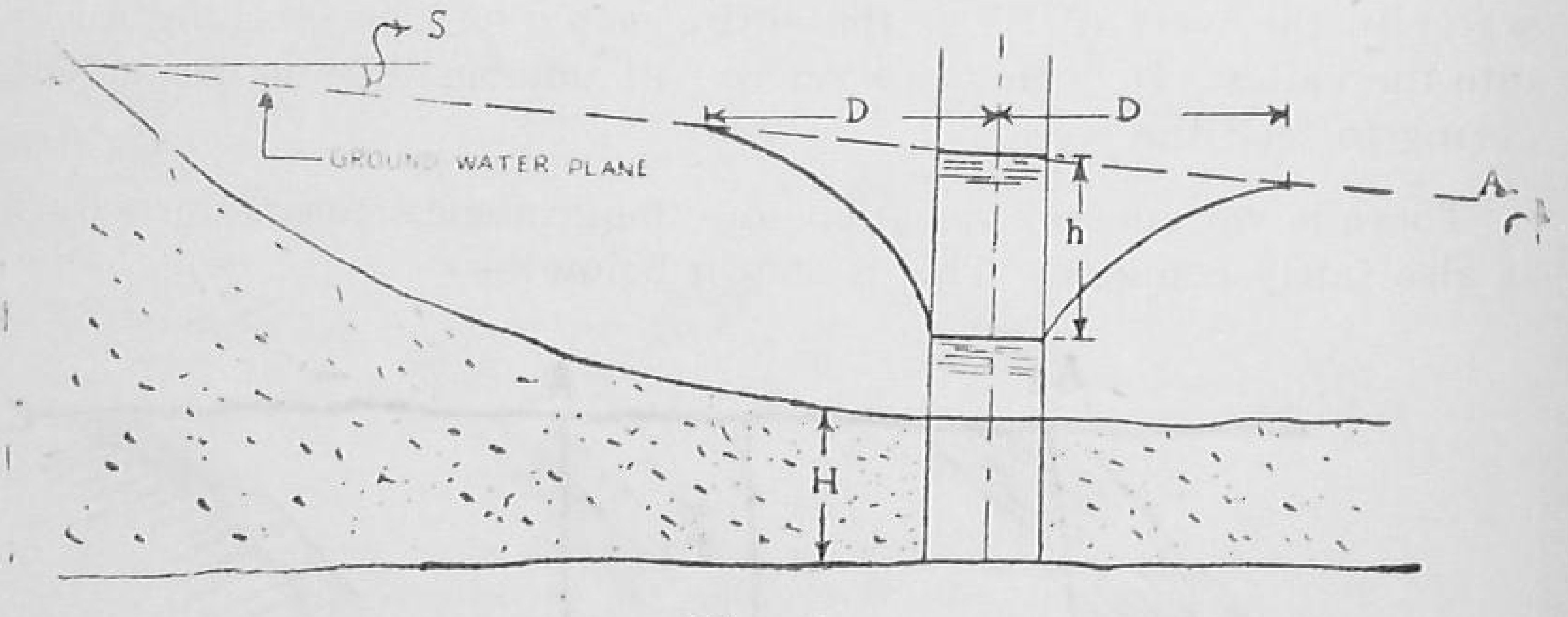


Fig. 28

The dotted line AA indicates the ground water plane parallel to the direction of the flow. This line will be gradually sloping down in the direction of the flow. A well has been sunk to a depth of H ft. in the porous water bearing strata. Before pumping, the water level in the well would rise upto the ground water plane AA. As soon as pumping starts, water begins to flow into the well from all sides through the porous medium. This flow could be imagined to take

place through a series of small pipe passages filled with porous materials which connect the wells to the source of water supply. The flow of water through the pipe passages is governed by the laws of "Flow of water through porous medium", and when such a flow is induced, there is a loss of head in overcoming the frictional resistance. The natural result is the lowering down of the water level which is known as "depression". The discharge from the well would depend upon the depression head 'h' *i.e.* the more the discharge, the more would be the depression head. If observations are taken of the ground water contours in the neighbourhood of a well from which pumping is being done, it would be seen that the effect of pumping occurs only within a certain distance 'D' on all sides of the well. Beyond that, the ground water contours remain unaffected. The width 2D is known as the tributary area of the well.

Now suppose the depth of the water bearing stratum tapped in a particular well is H. The cross-sectional area of the porous medium feeding this well would be 2DH. The quantity of water flowing towards the well would, therefore, be equal to $2DH \times$ the velocity of flow of the ground water. The velocity would depend upon (i) the slope S of the ground water plane AA, and (ii) modulous of the soil M and is given by $V = M \times S$. The modulous of a particular soil mainly depends upon the size of the soil particle *i.e.* the bigger the size of the particle, the less would be the resistance to the passage of water and more would be the flow. It is defined as that quantity of water which would be transmitted in a unit of time through a cylinder of soil, one unit in length, one unit cross-sectional area and under one unit head. Thus the discharge flowing into the well is $Q = 2DH \times MS$*. This quantity is the maximum permanent yield from the well. If the draught from the well is in excess of this quantity, the well in time will fail *i.e.* for a definite quantity of water to be drawn from a well for an indefinite period of time, there must be an actual flow of ground water towards the well. "The concept of a static lake of ground water is erroneous because it is inconceivable to think of an accumulation of ground water which is not discharging at some point, either at the surface or by deep leakage, and is not receiving water from some source, usually the margin of the basin. The water table is universally an inclined surface sloping in the direction of ground-water movement, and wells pump from moving ground water"†.

There is yet another limiting factor. Even though the above discharge may be available from the ground water flow, the discharge per sq. ft. of sand area exposed into the well is not to exceed a certain

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*P. 101-Hydrology and Ground Water by J.M. Lacey.
†P. 384-Ground Water by F.G. Tolman.

maximum discharge known as the 'critical discharge' as otherwise the sand particles will start moving and the well, in course of time, may collapse. The critical discharge per sq. ft. area of the sand is that provided by an infiltration head which causes a velocity sufficient just not to move or blow the sand into the well. This head is called the 'critical head'. The critical discharge depends upon the effective size and specific gravity of the sand. If during excessive pumping, the critical head is exceeded, sand particles start moving and are blown into the well. For maintaining the increased discharge during pumping, if the sand is removed and the well cleaned, fresh sand will be blown into the well. This continuous process of sand removal may result in the formation of underground cavities and either the earth round the well may collapse or the masonry cylinder may sink dangerously. Therefore, for a percolation well, no attempt should be made to draw water more than its safe yield *i.e.*, which would neither allow the critical head to be exceeded nor cause permanent lowering down of the ground water level.

In the above treatment, it has been presumed that the flow of water into the well is radial and the well steining is permeable. The wells in the alluvial plains have generally impermeable steining and the flow of water into the well is from the bottom. In such a case, the discharge into the well would depend upon the sand area exposed at the bottom *i.e.*, its diameter and would be independent of the depth of the well.

3. **Siting of Percolation Wells.**—For any major scheme of well sinking, it is difficult to estimate with any fair degree of accuracy what yield would be available in a particular area. The best method would be to collect all the necessary hydrological data from the existing wells if there be any, or to sink trial bores and take the necessary observations. Details of strata met with in the excavation of wells or sinking of trial bores should be collected and studied. Pumping operations could be performed on these wells and the following hydraulic observations* taken:—

(i) The quantity of water pumped in a given time *i.e.*, the discharge.

(ii) The "draw-down" or lowering of the water level. If the water level becomes stationary, against a certain rate of pumping, it would mean that the supply from the ground water flow is sufficient to meet the requirements of the pump. By increasing the pumping rate, the approximate supply available in the ground water flow could roughly be estimated. The draw-down necessary to produce a certain discharge from the well, is a direct function of the permeability of the water bearing formation

*Ground Water by Tolman, p. 381.

i.e., the more permeable is the water bearing stratum, the less would be the draw-down for the same discharge.

(iii) The discharge per ft. of draw-down from which the yield from two or more wells could be compared.

(iv) The static level to which the water rises after pumping is stopped. Overpumping is indicated if the water does not rise to its original level after pumping stops and safe yield is indicated if the recovery between pumping periods is complete.

(v) The rate of recovery which depends upon and consequently indicates permeability of the surrounding aquifer.

Having taken the above observations, the ground water contours could be ascertained from the records of water levels in the wells or the trial bores. If old records are available, it would be interesting to note if there has been any gradual change in the level of the water table. There are sometimes wide variations in water level during different parts of the year and often from year to year. It would be desirable to determine the contours from the records of one of the driest years, should these be available. The line of steepest slope would give the general direction of flow of water. The slope of the ground water plane having been determined, the velocity of the ground water flow could be ascertained from the size and the grains of the water bearing strata. From this, the discharge available in the ground water flow could be roughly estimated. In absence of detailed data, the safe yield in an existing well could also be determined by ascertaining the time it takes to recoup to its original water level after water has been pumped from it for some time. For example, suppose 30,000 gallons of water have been pumped out of the well in three hours and it takes 9 hours to recoup to its original level, the actual in-flow of the ground water into the well would be at the rate of 30,000 gallons in 12 hours *i.e.*, 2,500 gallons per hour. After the safe yield has been approximately gauged, the number of wells to be installed could be fixed. This would depend upon the extent of water bearing strata available and the installation would consist of a line of wells at right-angles to the general direction of flow of ground water, care being taken that the wells do not mutually interfere *i.e.*, the tributary areas of the respective wells do not overlap. The following table shows the mutual interference of a number of wells in a row:—

Distance apart of the wells (in ft.)	Mutual interference (per cent
100	65.8
200	45
400	24
600	14
1000	6.4

In hilly and steeply sloping country, broad open valleys with easy gradients furnish better sites for wells than narrow valleys with a rapid fall. The reasons are two-fold. The first is that due to steeply sloping land, most of the water escapes as free-water and does not get time to penetrate under ground. The second reason is that due to high velocity of surface water, large portions of water bearing soil are washed away thereby reducing the depth of water bearing formation. Sinking of wells in valleys with easy slopes and at junction of valleys will also be quite successful.

4. **Well Sinking.**—For sinking a well, circular pit slightly bigger than the proposed diameter of the well is dug into the ground. Digging is continued to within a foot or two of the sub-soil water or soft soil, like sand, whichever is met with earlier. The well curb made of wooden planks or boiler plates is then bedded in a horizontal position. The function of the curb is that, during the process of sinking, it takes the strain, due to unequal settlement of the well ring, resulting from the soil underneath the well steining on one side offering greater resistance to sinking than that on the other side. A timber well curb generally consists of several plank pieces of hard wood 6″ tc 18″ in thickness nailed together in a circular form as shown below:—

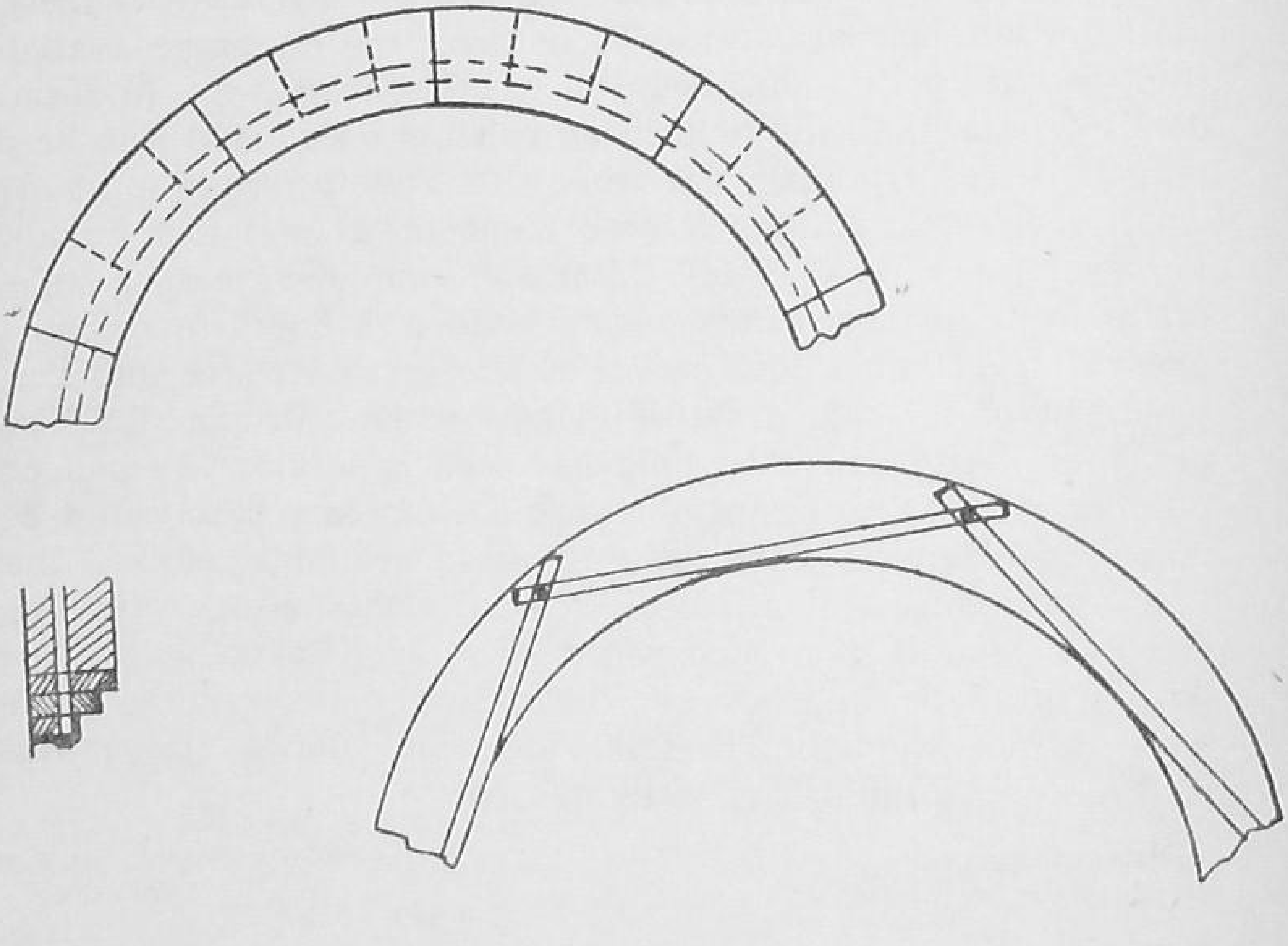


Fig. 29

The joints in the planks overlap each other and the bottom-most plank-ends in a cutting edge made of angle iron. As will be described later, during the sinking operation, it sometimes happens that due

to some obstruction, the well does not sink even after the whole earth and debris under the curb have been cleared off. When this occurs, the well along with the curb remains suspended. It is held together only by the strength of the mortar and there is the danger that the curb may get detached from the well or the bottom few feet of masonry may get separated from the remaining portion of the well. To prevent this, 5 or 6 tie-rods about 8 ft. to 10 ft. long are fixed vertically in the well curb and embedded in the well steining. These tie-rods have screwed ends and carry iron plate washers with nut, each iron plate joining the two adjacent tie-rods. When the nuts are tightened, the entire bottom masonry of the well along with the curb is held together and there is no risk of one getting detached from the other. In deep and bigger wells, the curbs are often subject to considerable strain and in such cases, it is desirable to make these of reinforced cement concrete or of thick steel plates.

After the curb has been bedded in position, the masonry ring of necessary thickness is constructed over it in a height of 4 to 6 ft. This masonry is kept constantly watered and allowed to set properly. After it has set, the earth and sand from within the ring are scooped out by manual labour. Ordinary small bamboo baskets are used for the purpose and when the depth increases, the earth in the baskets is lifted above by means of a rope working on a pulley. With the removal of sand and earth from within, below the well ring, the latter would gradually sink by its own weight. To facilitate sinking, wooden planks are placed across the top of the well. The planks project beyond the masonry ring and the projections of the plank are loaded with sand bags and other heavy articles. After the whole length of ring has been thus sunk, another layer of masonry is constructed and the process repeated. When sub-soil water starts coming into the well, it would not be possible to remove the excavated material by means of baskets and use has to be made of bull or some other kind of hand-dredger. "Jham" is a very common implement used in the alluvial plains.

In wells sunk in the manner just described, it is obvious that the more the thickness of the steining, the more would be its weight and the easier would be for the well to sink. The resistance offered by the sides of the wells in sinking is known as the "Skin Friction" and the thickness of the well is a direct function of this and the depth of the well, since on the latter factor would depend the total surface area offering the resistance. As a rough rule, "the sinking effort in cwts. per sq. ft. of the skin of the steining is one thirtieth of the depth below the ground surface the well has to be sunk. From this the thickness to be given to the well steining could be deduced." *If this thickness is provided, the need for artificial loading by sand

*P. 112 of Hydrology and Ground Water by Lacey.

bags as described above will not arise and the sinking operation in the long run may prove to be much cheaper.

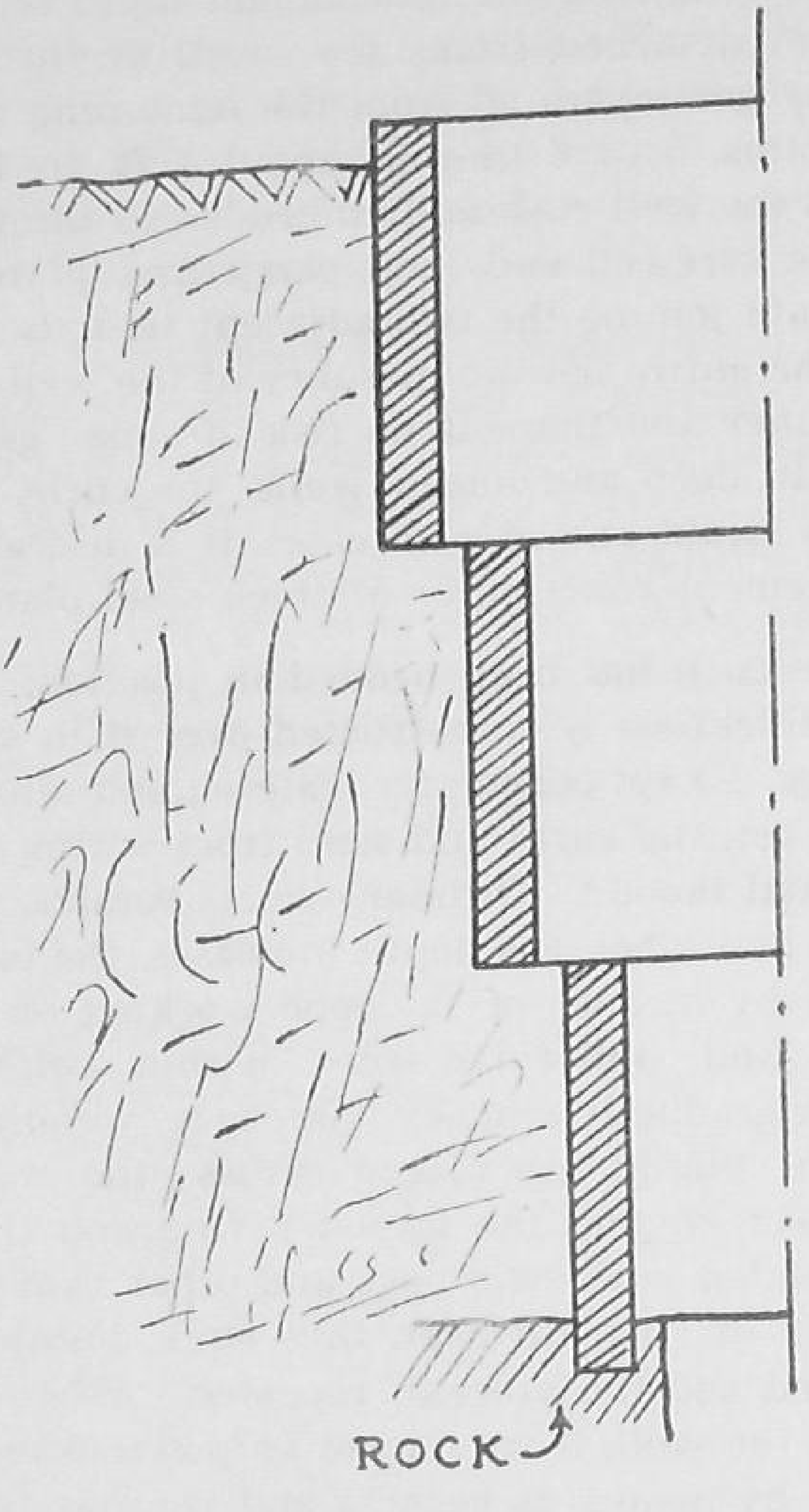


Fig. 30

Whenever the sub-soil is hard, and well sinking in the above manner is not possible, the well is excavated right upto the proposed depth. The sub-soil water which rushes into the well is pumped out by means of power pumps and construction of the well steining started right from the bottom. In such a case, it is obvious that a well curb is not needed. To avoid deep vertical cutting and consequent danger to workmen, the excavation and also construction of the well steining could be carried out in steps as shown in the sketch. Starting with the biggest diameter at the top, each step of the masonry ring is finished off before excavation of the well in the next step is started.

5. Cavity Wells.—So far we have dealt with only that class of wells in which supplies are drawn from the free ground water below the water table held up in the porous formation overlying a layer of clay, rock or some other impervious strata. In the alluvial plains, very often it happens that below a layer of clay, there is again a thick layer of sand or other porous material. The two porous strata separated by clay are directly connected at their upper edges and thus form a continuous medium (see sketch below).

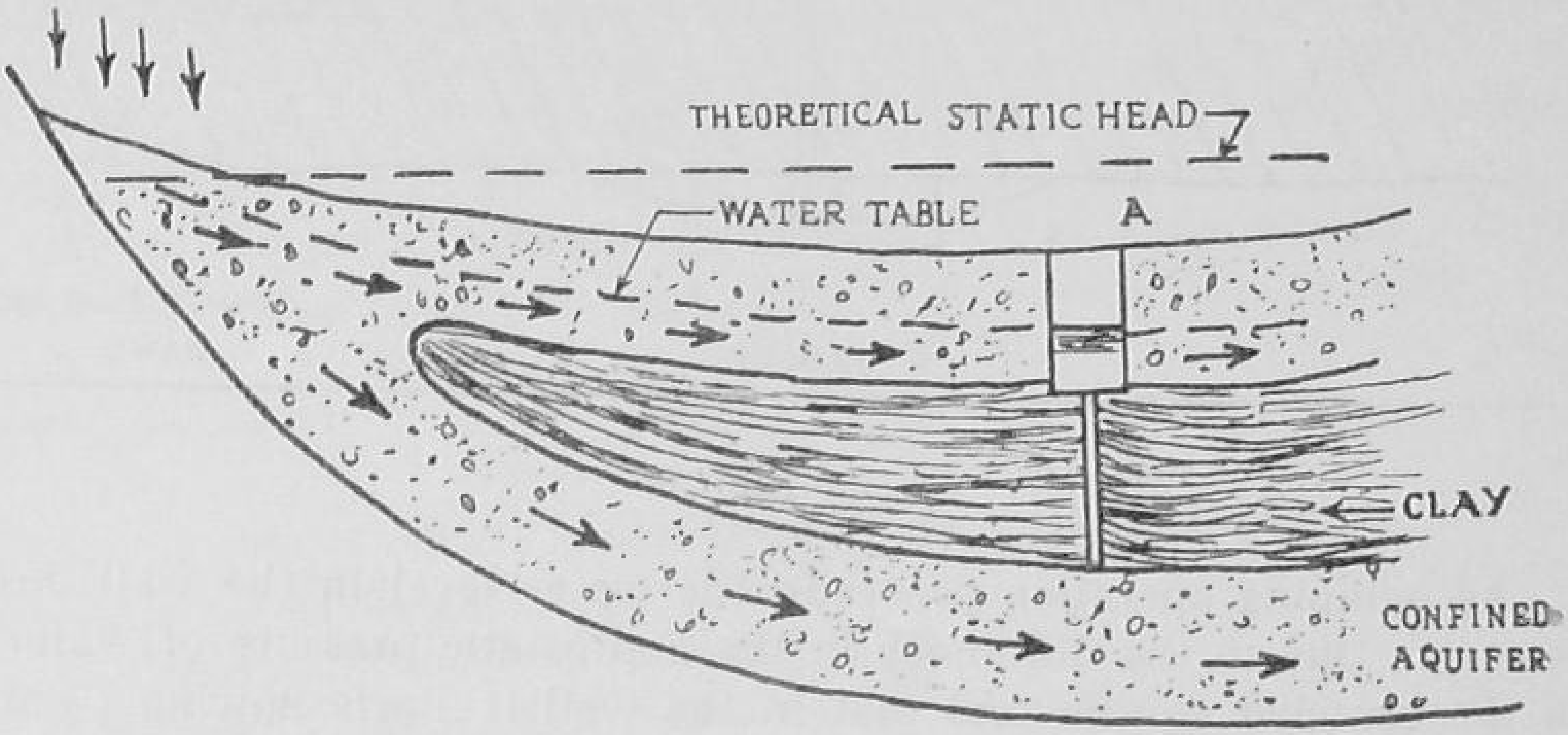


Fig. 31

Water penetrating in the upper layer of the sandy soil, finds its way into the lower porous strata also and is held up there between the intervening layers of clay. Similarly, there may be a third or even a fourth layer of some porous water bearing formation each separated from the other by layers of impervious material like clay, rock etc. Water contained in the lower strata of sand will obviously be under pressure and would tend to rise to the rest water table if a hole is drilled into it through the intervening clay. The water thus held up is called "confined water". As different from percolation wells, there is a type of wells, which draws its supplies from the confined ground water below a confining formation, and cavity wells belong to this class. In the above diagram "A" represents a cultivators' well with its steining carried right into the clay formation. The steining is also impervious to water which in consequence excludes all supplies from the upper aquifer. If a pipe is sunk through the intervening layer of clay into the confined aquifer, the water in the pipe would tend to rise to a level corresponding to the hydrostatic pressure at that point. Now let us examine what happens if water is pumped out from such a well.

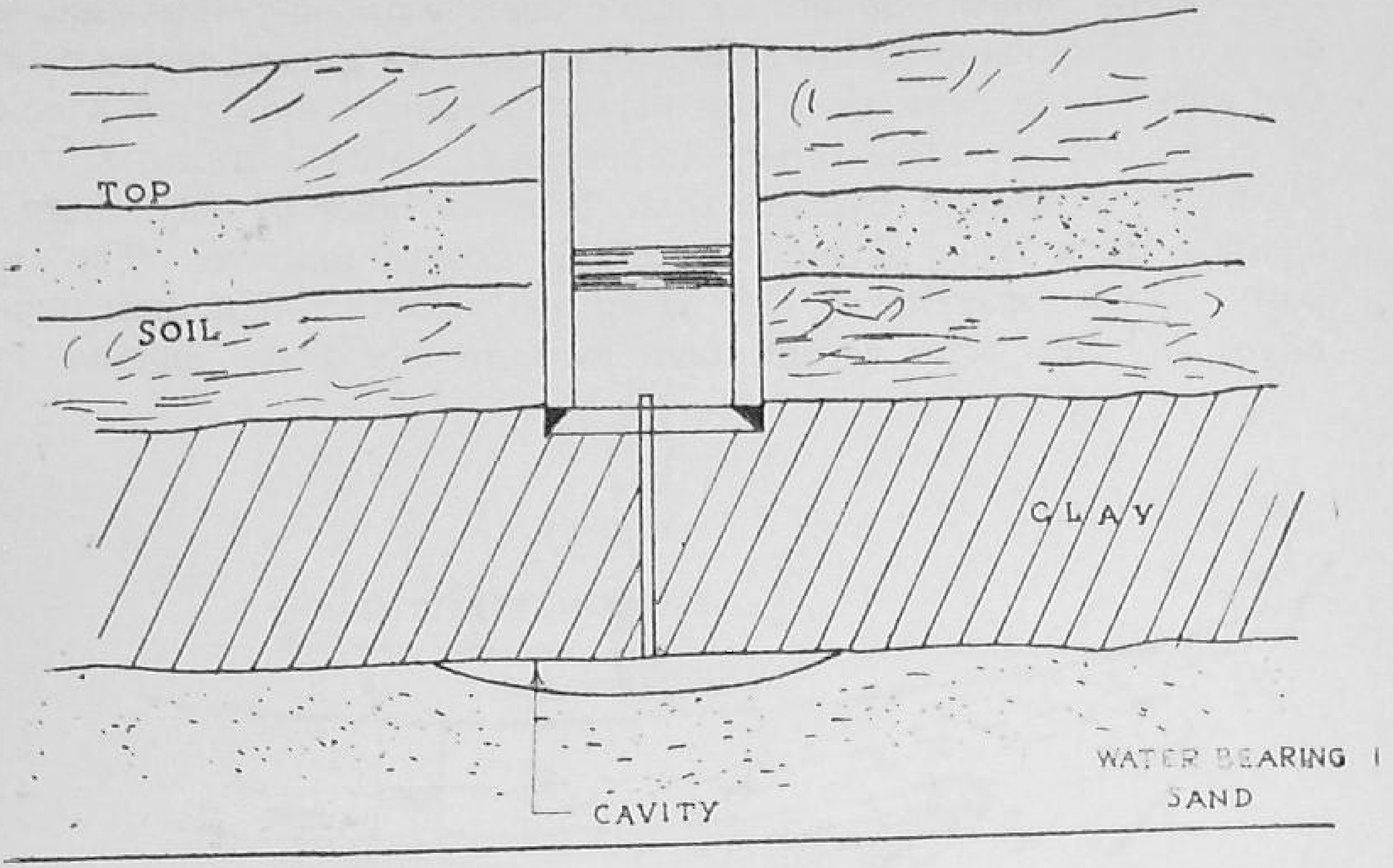


Fig. 32

As pumping operation proceeds, the water level in the well depresses. Due to the difference in the hydrostatic pressure of water in the confined aquifer and that in the well it starts flowing from the former to the latter. As the rate of pumping increases, there is a simultaneous increase in the depression head, which in turn, causes an increase in the discharge through the pipe. If this exceeds the critical discharge of the formation constituting the confined aquifer, sand will be blown out into the well through the pipe. This sand is cleared off. As pumping proceeds and if the well is deep enough to accommodate the corresponding depressed water level, more and more sand would be pumped out, resulting in the formation of a cavity just underneath the clay. The clay when it is sufficiently firm and has the required strength to bear its own weight, and the weight of the masonry cylinder, acts as a roof for the cavity and a much larger area of water bearing formation thus becomes exposed to the bottom end of the pipe. The sand continues to be drawn out till the surface area exposed in the cavity is sufficient to give a total discharge equal to that which is being pumped out without its exceeding the critical discharge per sq. ft. When no more sand comes out, the cavity is fully developed and the well is ready for use. The pipe should be sunk to such a depth that its lower end is in flush with the bottom of the clay formation. In the well also, it should just project out so that much "head" is not lost. For the sand to be forced out into the well from the lower aquifer, it is necessary that a certain minimum velocity is induced in the bore pipe. This determines the minimum depth of water, below the water table

which would ensure the successful development of a cavity well. In general, the head required for ordinary depths is:—

(1) 10 ft. to obtain velocity in the tube or a bore well to move the sand and give the required discharge.

(2) 5 ft. in the well for bailing apparatus to work in.

(3) 3 ft. for the fluctuations in the ground water level. That is, the normal depth required is 18 ft.*

In a well of the type described above, it is obvious that the diameter of the well has no bearing on the yield from it as this would depend only on the size of the cavity. The average depth of the cavity is about 3″. The quantity of sand removed gives an indication of the size of the cavity. The essential condition for a cavity well is that it should have a strong and dependable roof. If the stratum which is above the cavity is thin or friable or sandy, it is very likely to give in and the construction of a cavity well would not be feasible.

If in the above arrangement, the suction of the pump is directly connected to the pipe, cavity in the lower aquifer could be developed irrespective of the water level in the parent well. Since the rate of pumping exceeds the critical discharge of the formation constituting the confined aquifer, sand will be blown out. This is the principle on which cavity tubewells are sunk. For this purpose, it is not necessary to have a well at all except for installing the pumping machinery.

6. **Increasing water supply in a well**.—Where an existing well gives a short or inadequate supply, the simplest and most economical procedure to increase the yield is to deepen the well and thus tap additional water bearing surface if there is possibility of obtaining any radial flow. Increase in its diameter may also give a larger cone of depletion and hence an increased supply. In the alluvial plains, where possibility of tapping lower aquifers always exists (as in the case of Mota or Cavity wells described above) boring is almost invariably successful, whenever it is required to augment the water supply in an existing well. In the rock well areas, as distinguished from the alluvial areas, the hard rock at the bottom is excavated by local appliances and explosives available. As there are numerous joints and fissures in the hardest rocks, it is possible that a water bearing fissure may be tapped in deepening the well thereby connecting it to a remote underground reservoir of decomposed or disintegrated rock. The well is also widened to obtain greater storage. Where an air compressor is easily available, holes are

*Hydrology and Ground Water by I. M. Lacey, p. 141.

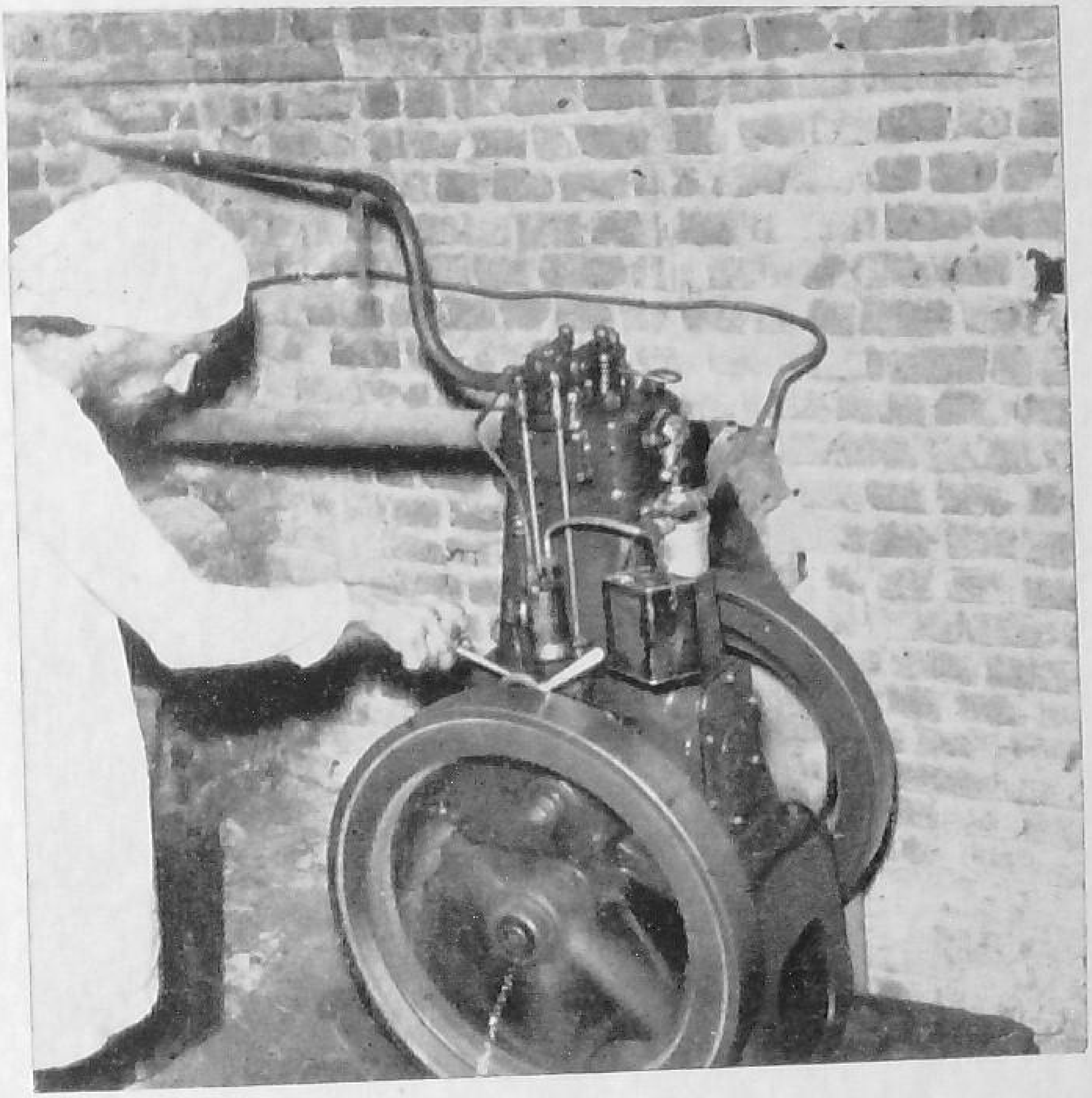
drilled in the rocky bottom of the well and it is blasted by means of gelegnite. This considerably facilitates deepening.

In tracts, where there is a general deficiency of rainfall, and whatever surface run-off is available easily escapes off as free water due to peculiar topography of the land, the best way to augment the ground water supplies would be to obstruct the free and quick flow of the surface water by the construction of suitable embankments or sub-surface dams across the various drainages. The wells situated immediately below a tank are known to have far better water supplies than those distantly removed from it. In many places, where a road causeway exists, it arrests the quick flow of surface water and charges the sub-soil with plenty of ground water --the result is an almost immediate increase in well water supplies in the vicinity.

7. Prevailing practice of lifting water.—Raising water for irrigation has been practised for many centuries in this country. The "Dauri", "The Don" and the "Dhenki system" are still very common in the Northern States particularly in Uttar Pradesh. In the "Dauri" system, two men lift water to a height of 3 to 4 ft. by means of a swinging basket. Approximately 12 gallons per minute are lifted by this method. In the "Dhenki system" use is made of the principle of a lever with a suspended fulcrum and a counter-weight. A metallic bucket hangs from the long end of a pole, levered on a Y-shaped log of wood. The operator lowers the bucket by pulling at the rope, the bucket dips into the well and the counter-weight rises up. Water lifted by this method is about 5 to 8 gallons per minute from a well of normal depth. The "Don" is an improvement on the "Dauri" system, but uses the principle of "Dhenki". The receptacle is an oblong bucket of galvanised iron sheets levered at an intermediate point. The long arm is suspended by means of a rope tied to one end and is made to dip into the water. The counter-weight pulls the receptacle upwards and water is lifted in it by about 4 ft. The devices most commonly used now are operated upon by animal power. "Persian Wheel" and "Mote System" come under this category. In the 'Mote System', a bullock or a pair of bullocks draws water from the well in a big leather bucket known as the 'Mote'. While drawing water, the bullocks take advantage of the downward slope of the ground on which they move. The Persian Wheel consists of a horizontal wooden wheel about 10 ft. in diameter with cogs projecting. The wheel is supported on a vertical shaft which rests on a wooden bearing and is held in position by a wooden beam. The horizontal wheel rotated by an ox or a pair of oxen, is geared to a vertical wooden wheel on horizontal shaft to which is also fastened a large vertical wheel supporting metallic buckets or

The Persian Wheel driven by a Camel.

Traditional irrigation device of the region is this animal driven Persian Wheel capable of pumping shallow well water at a rate to irrigate about half an acre daily.



Power for the new pump is furnished by the six horse power diesel.

Water for India's campaign to grow more food comes also from this newly constructed well.

earthen jars with which to raise the water. The Persian Wheel is able to lift about 1500 to 2000 gallons per hour from wells of normal depth.

8. Modern Pumping Methods.—Pumping machinery of high efficiency is now most commonly used for lifting water for irrigation purposes. The economics of lifting water by means of power pumps has been discussed in a later Chapter (see Ch. VIII). What follows would, therefore, be confined to a description of the various types of pumps, their operation, quantities of fuel required etc.*

(i) Power Requirements of Pumps.—Mechanised power is defined as the time rate of doing work. Thus to lift 2 cubic ft. of water (125 lbs.) through a vertical distance of 1 ft. each second would require 125 ft. lbs. per second, provided the lifting devices were 100 per cent. efficient. If the pumping plant efficiency were only 50 per cent., it will require 250 ft. lbs. per second, thus providing for a loss of one half of the total required power in overcoming friction, in generating heat etc. The unit of power most commonly used is the H.P. which is 550 ft. lbs. per second or 33000 ft. lbs./minute. One H. P. would lift 1 cubic foot second (C.F.S.) a vertical distance of 8·8 ft. with an efficiency of 100 per cent. *i.e.*,

$$\text{H. P.} = \frac{1 \times 62 \cdot 5 \times 8 \cdot 8}{550} = 1$$

The "Water Horse Power" is defined as the power theoretically required to lift a given quantity of water per second to a specified height and is given by $\text{H.P.} = \frac{62 \cdot 5qh}{550} = \frac{qh}{8 \cdot 8}$

where H. P. = Water Horse Power.

q = Cusec.

h = vertical lift in feet

If capital Q is measured in gallons per hour we have

$$q = \frac{Q}{3600 \times 6 \cdot 25}$$

$$\text{and H.P.W.} = \frac{Q}{3600 \times 6 \cdot 25} \times \frac{h}{8 \cdot 8} = \frac{Qh}{1{,}98{,}000}$$

The H.P. delivered by an electric motor or an engine to the shaft it turns is known as the B.H.P. The ratio of the useful water horse-power delivered by a pump (the output) to the brake-horse power (the input to the pump) is defined as the pump efficiency. In irrigation pumping the output would thus equal the water horse power. Pumping plant efficiency is defined as the ratio of the power output to power input and would thus be equal to

$$\frac{qh}{8 \cdot 8} \text{ horse-power input}$$

*Extracted from Irrigation Principles and Practice by Orson W. Israelsen, Ph. D.

(ii) Pumping Characteristic.—In order to use modern pumps most profitably to obtain irrigation water, it is essential to select pumps well adapted to the particular conditions of operation and thus obtain relatively high efficiency. If the rate of pumping is appreciably less than that for which it is designed and the head excessive, a low efficiency would result, and *vice versa*. The inter-relations between speed, head, discharge and H.P. of a pump are usually represented by curves designated as "characteristic curves". Knowledge of the characteristics of the pump enables the manufacturer and irrigator to make adaptations of the pump to the operating conditions and thus attain relatively high efficiency at low operating cost. The following characteristic curves for a particular pump will make this clear.

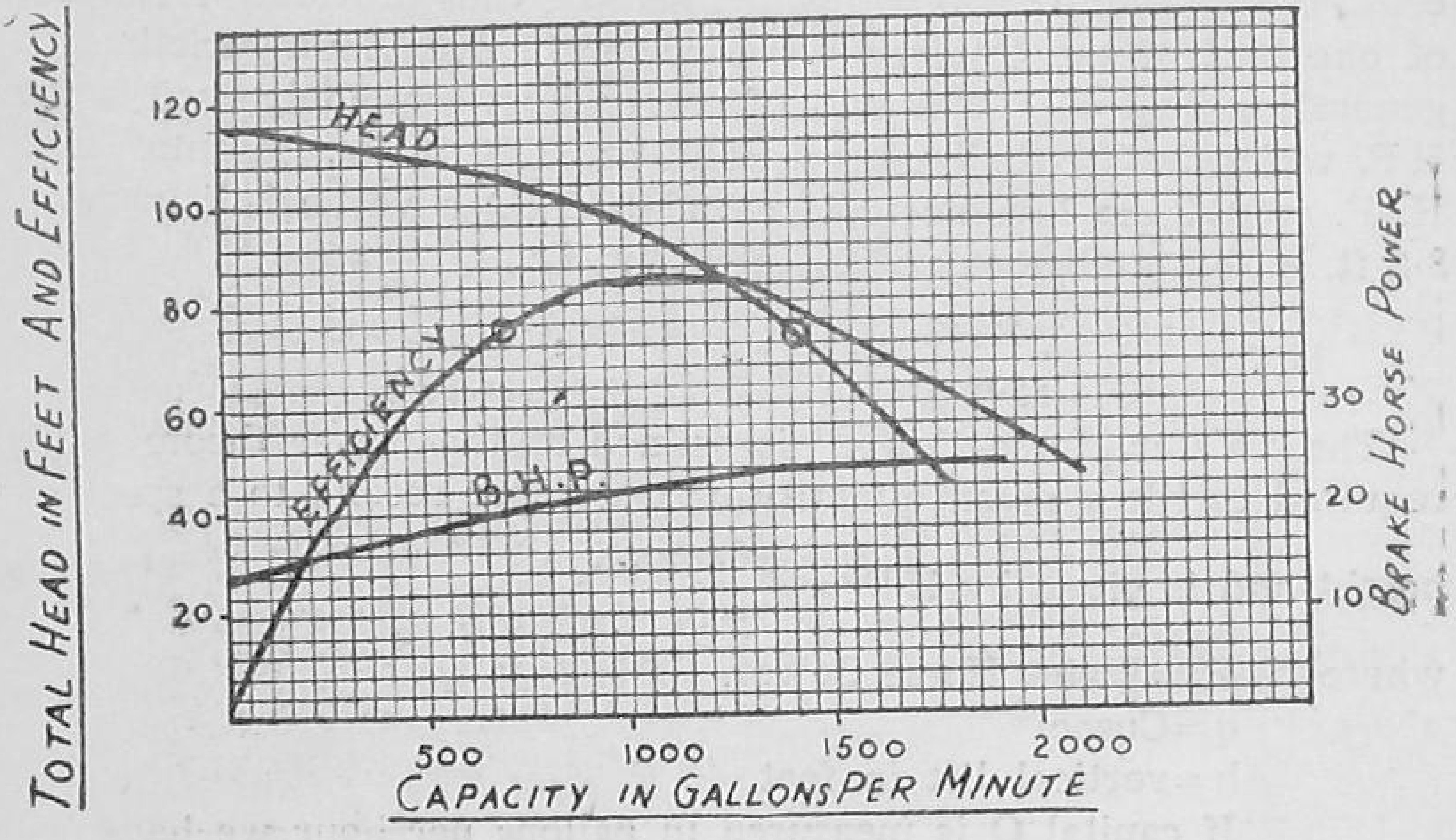
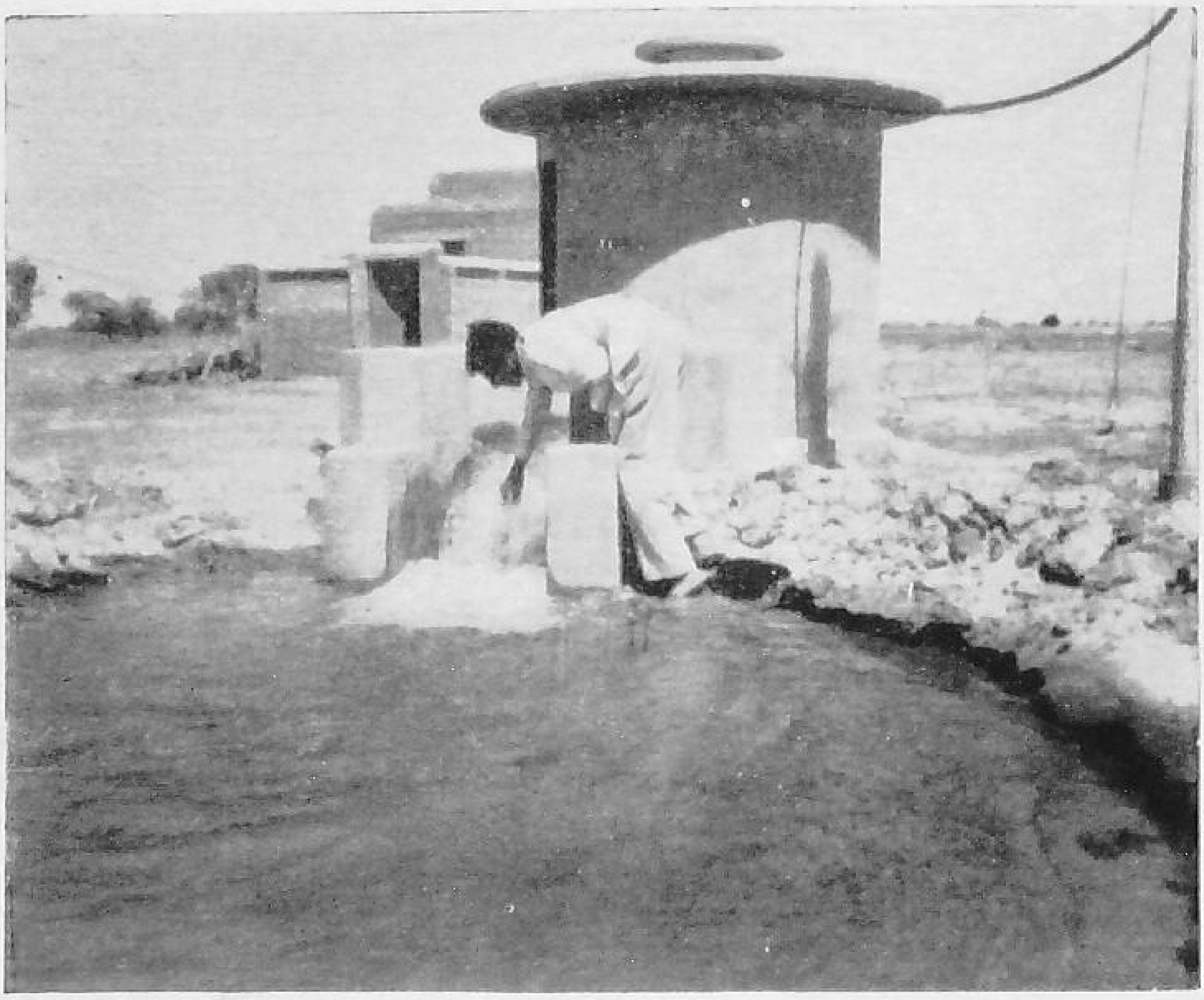


Fig 34

For quantity ranging from 700 gallons upto 1440 gallons per minute and heads ranging from 105 ft. down to 65 ft., the efficiency of the pump will be equal to or greater than 75 per cent; also it will attain the maximum efficiency of 82 per cent. at quantities from 1100 to 1200 gallons per minute and at a total head of 90 ft.

(iii) Types of Pumps.—The pumps most commonly used may be grouped into four types namely; (i) Centrifugal, (ii) deep well turbine, (iii) screw type or direct flow and (iv) reciprocating or plunger pumps. Types (i) and (iv) are very commonly used and do not need any special mention. In order economically to obtain water from deep small diameter wells, modern deep well turbine

View of tubewell pumping station and pump attendant's house.



The Tubewell operator plunges his hand into the life-giving water from the tubewell.

centrifugal pumps (item ii) have been developed and greatly improved. The rotating impeller is built on a vertical shaft within a compact bowl, the entire unit being known as a stage. For high lifts two or more stages are placed in series near the bottom of the well. The pump is driven by an electric motor or other prime mover set at the ground surface and connected by long vertical shaft. Being submerged, the deep well pumps have the advantage of requiring no priming and of meeting rather wide fluctuations of water surface without necessitating the re-setting of the pump. They have the disadvantage of inaccessibility of operating parts and consequent difficulty of inspection. The screw type of pump consists of rapidly rotating screw like impellers fastened securely to a shaft mounted on bearings within a pipe column. The action of screw type pumps in water is sometimes compared to the action of a rotating fan in air—both drive the surrounding fluid forward by the rotation of inclined planes. Screw lift pumps are used on both horizontal and vertical drive shafts.

(iv) Kinds and Costs of Fuel.—For small pumping plant, electricity, gassoline, crude-oil and coal constitute the more important sources of power. The cost of fuel from which to obtain the necessary power influences the total cost of pump operation and hence the selection of the plant. The following quantities of the various fuels may be adopted as essential to develop one H.P. hour:—

1. Electricity $\frac{9}{10}$ K.W.H.
2. Gassoline or distillate $\frac{1}{6}$ gallon
3. Crude oil 6 lbs. (9 lbs = 1 gallon)

TUBEWELLS

1. It has been stated earlier that there is a class of wells which draws its supplies from confined water below a confining formation as distinct from the free ground water below the water table (para 6). Like the cavity wells, tubewells also come under this class. In the Northern plains in which alluvial deposits extend to great depths, beds of clay inter-stratified with sand are a fairly common occurrence and in most cases, a tubewell can be successfully installed.

In the Indo-Gangatic plains of Northern India, the rivers in their upper reaches flow with great velocity and carry with them large particles of sand, gravel and boulders. Consequently, the alluvial deposits in these regions consist of these materials only and little or no trace of clay is to be found here. As we go lower down the

valleys, the river slope becomes flatter where large particles of sand and gravel cannot be transported in suspension nor can be rolled along the bed. Finer deposits like clay, come into prominence and occurrence of alternate layers of sand interspersed with beds of clay becomes a feature of the sub-soil formation in these regions. As we go still lower, we reach the delta zone. Here the rivers become so flat that they carry very little sand in suspension and presence of clay particles predominates. Consequently, the sub-soil formation also changes considerably and clay becomes its main constituent with traces of sand here and there and that too at great depths. The coarseness of the sub-soil will thus be an indication of the particular stage of river flow at which it was deposited.

In order to draw large supplies of water, wherever water bearing strata of great depths are available including the lower aquifers which are separated by layers of clay, a tubewell can generally be installed with success. The flow in a tubewell is radial and water enters it through perforations in the pipe, which coincide with the water bearing strata. The various methods of boring now commonly in use and the types of tubewells will be described in the paragraphs that follow.

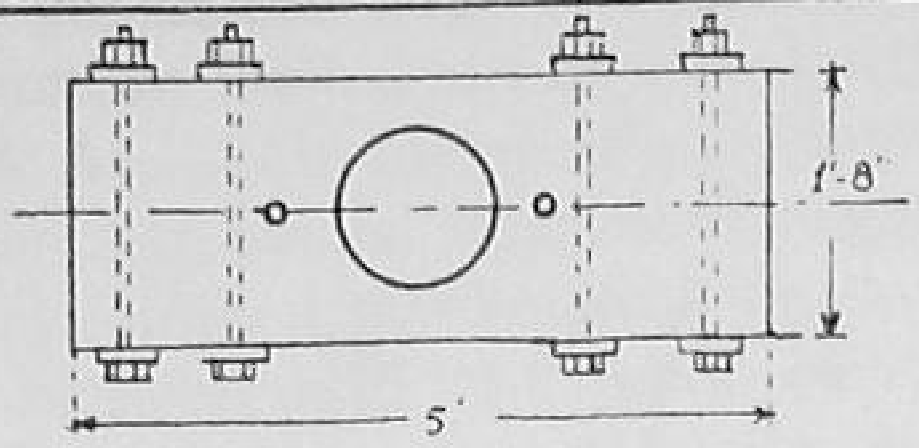
2. **Methods of Boring***.—Boring systems can be divided into 3 main classes *viz.* percussion boring by which a hole is made into the ground by reducing the strata to powder, water jet boring which makes a bore by washing out the strata and rotary boring by which a hole is drilled into the strata and a solid core obtained.

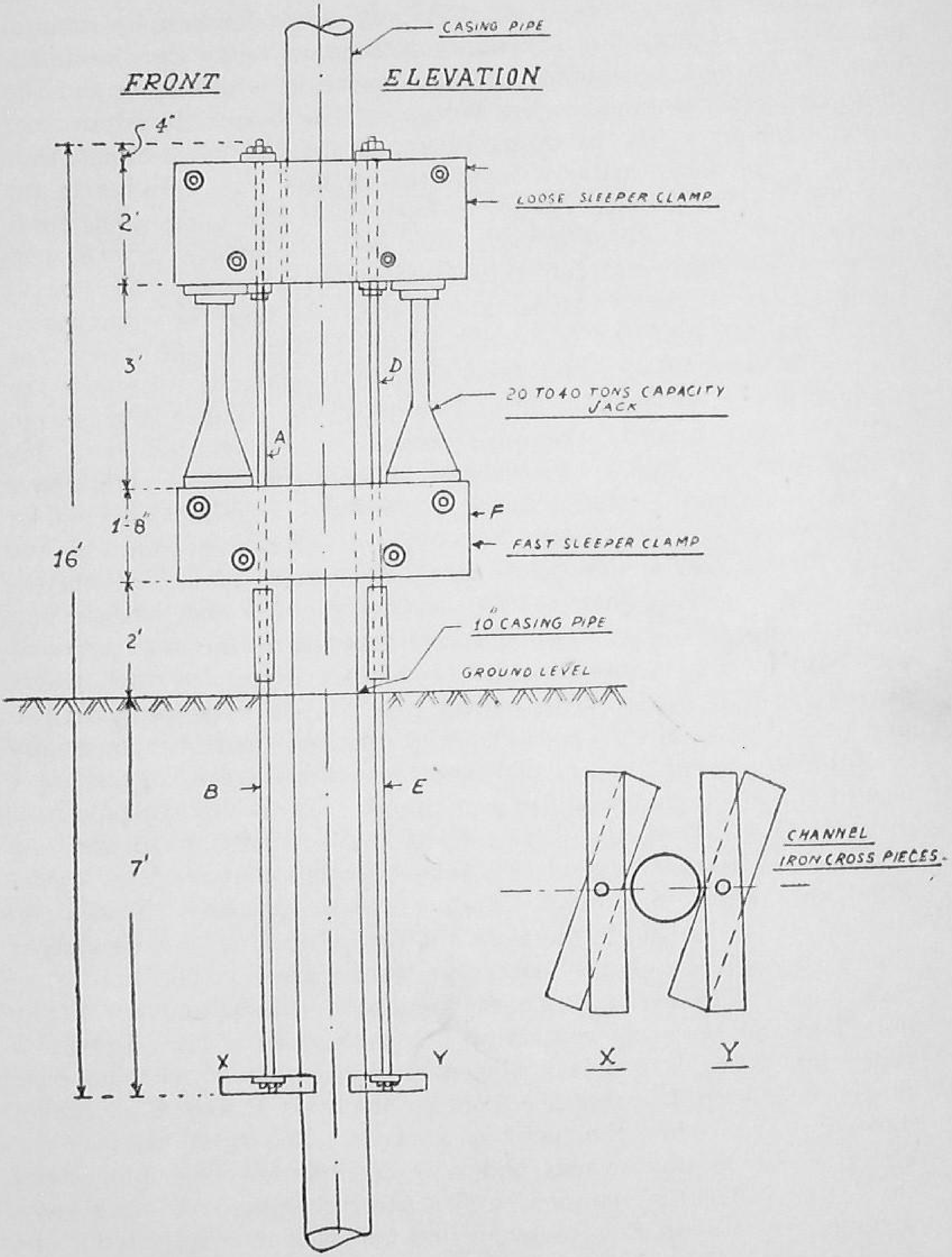
(i) Percussion rope boring.—A pit about 6 to 8 ft. in diameter and 15 to 18 ft. in depth, is dug at the site where boring has to be done and the boring tube is lowered into it with a cutter shoe fitted to its bottom. In the case of very deep borings or borings in clayey strata, the bore is started with a larger diameter boring tube and carried down till the friction on its outer surface makes it difficult to carry on the boring any further. A slightly smaller sized boring tube is then lowered inside the larger sized tube. If necessary, a still smaller size of the boring tube is lowered inside the inner tube when the friction of the latter also becomes excessive. Thus the diameter of the bored hole is successively reduced as the conditions demand. This system of lowering boring tubes one inside the other is called "Telescopic boring". After being lowered in the pit as described above, the boring pipes are clamped in position with wooden clamps and partly filled with water. Water is necessary for boring operations and must be poured from the top till water is struck in the bore hole. Actual boring is done with a tool known as "Sludger" or "Sand Pump". This implement is a steel pipe of about $\frac{1}{4}''$ in thickness varying in size and diameter according to the size of the well to

*Extracted from Hand Book of Tubewells by K. D. Sanwal.

be bored. It is about 2" less in diameter than the casing pipe and is about 8 to 12 ft. long. A cutting shoe of hard steel or tungsten steel is riveted to the "Sand Sludger" at its lower end. A flap or ball valve is fitted inside the sludger pipe just above the cutting shoe. The sludger is suspended by a wire rope from a pulley on a tripod. The legs of the tripod are buried about 2 ft. underground to prevent their slipping or tilting. The sludger comes centrally over the boring tube and is worked up vertically up and down by manual labour or an engine with a cranking arrangement etc. On the downward stroke of the sludger the flap valve is forced open and the loose material pounded at the bottom of the bore hole enters into the sludger pipe. As the up-stroke begins the flap valve closes, thus retaining the loose material inside the sludger pipe. Owing to the vacuum created by the upward movement of the sludger, the loose material is also sucked upwards and gets thoroughly mixed with water. The loose material suspended in water enters the sludger pump on the downward stroke and at the same time the cutting shoe cuts some more of the material at the bottom of the bore. The process is repeated and more and more loose material collects in the sludger pipe. After about 30 to 40 strokes, the sludger is taken out and the loose material, retained inside it, is emptied out. The boring tube is clamped with sleepers at some convenient point above ground level and over the clamps, a platform is made which can be loaded with sufficient weight to force the boring pipe down as the sludger excavates a hole below it. A convenient and more efficient method of applying load to the boring pipes is the anchor bolt method *vide* arrangement shown in the sketch on the next page. A pit about 5 ft. in diameter and 7 ft. deep is dug at the spot, where boring is to be done. The boring pipe "P" is fixed centrally. B and E are two mild steel rods about 1½" diameter and 16 ft. in length. X and Y represent two channel iron cross pieces bolted to the lower end of the rods. Earth is filled in the pit and around the pipe upto ground level. The rods B and E are now firmly fixed and any attempt to lift them would be resisted by the weight of the entire filled up earth acting on the channel iron cross pieces. F is a fast sleeper clamp *i.e.* holds the bore tightly while L is a loose sleeper clamp through which the bore pipe passes freely. The mild steel rods pass freely through the fast sleeper clamp and are firmly bolted to the loose sleeper clamp on both sides of the sleeper. J represents 20 to 40 ton jack placed on either side of the bore pipe. Since the clamp L is rigidly fixed to the rods B and E, it cannot move upward when the jack is worked. The result is that the clamp F moves downwards and with it sinks the bore pipe, since the latter is tightly clamped to F. After the pipe has sunk some distance, the clamp F is loosened and the jack is re-adjusted. The former is raised up by the distance the pipe has sunk down and is

PLAN OF THE UPPER SLEEPER CLAMP





ARRANGEMENT OF ANCHOR BOLT SYSTEM.

FIG. 35

again clamped tightly to the bore pipe. The process is repeated and the pipe sinks more and more.

(ii) Rod Drilling.—This is similar to rope boring except that rods extend from the ground level to the boring tube instead of a rope. The rope boring system is used for very deep borings or borings in very hard strata. Boring rigs are constructed either of wood or wrought iron tubes with a head pulley block over which a wire rope is passed to support the boring rods and tubes.

(iii) Water Jet Boring.—At places where sufficient water is available for working a water jet plant, the hydraulic washing system is a very convenient method of boring through hard and tenacious soluble clays in which progress of boring with ordinary percussion methods would be very slow. In this system, water under pressure, pumped by means of an electric or oil engine power pump, is admitted into the boring pipe which has a chisel fitted to its lower end. The chisel has two nozzle orifices, one on either side, and when water under pressure is forced into the boring pipe it shoots out through the nozzles and rises into the annular space between the boring pipe and the casing pipe. In its upward movement, the water carries, with it, any material that may have been dislodged by the force of the water jet. This water eventually overflows into the settling tanks at ground level. The capacity of the pump should not ordinarily be less than 15,000 gallons per hour and should be capable of working upto heads of 200 ft. The whole arrangement is shown in the sketch on the next page. This method has certain disadvantages with regard to the recording of the various water levels and also the depths and samples of strata. A combination of the water jet and percussion boring generally gives expeditious results and incorporates the advantages of both the systems.

(iv) Rotary Boring.—By this method, it is possible to obtain great speed in the boring operations. It is said that as much as 1,000 ft. have been drilled by this system in 24 hours. The practical operation consists of rapidly rotating a column of pipe at the lower end of which is a cutting bit. In the bit are holes through which water with clay in suspension is sent under pressure. The fluid mixes with the cuttings and carries them upto the ground surface in the annular space between the drill pipe and the bore hole. The clay that is held in suspension muds up the whole of the hole and causes the pipe to turn more rapidly as there is practically no friction between boring pipe and the sides of the bore hole. The machine used for rotating the pipe is called "Rotary".

(v) Core Drills.—This system is generally used for rock boring and is very rarely used for boring of tubewells.

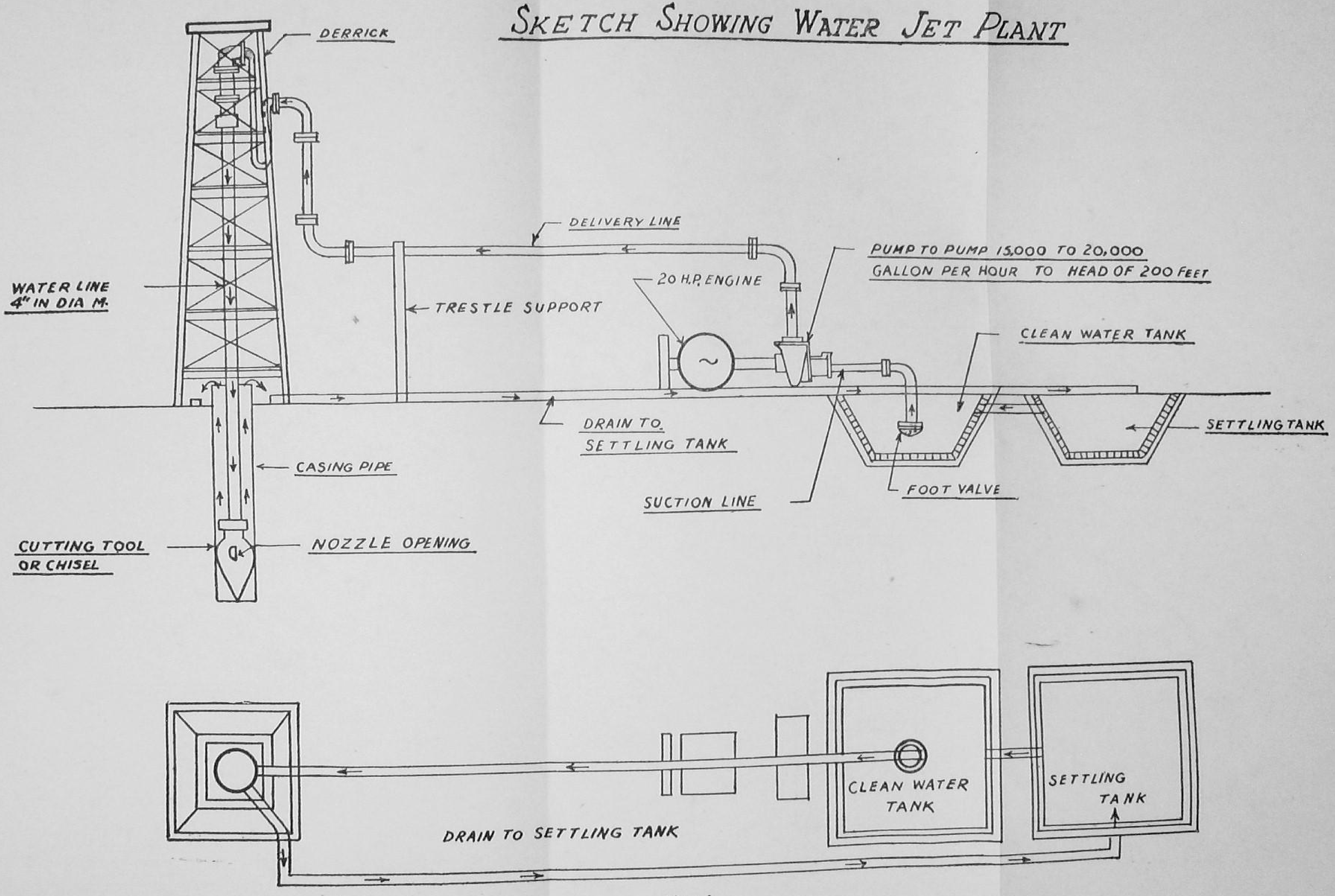
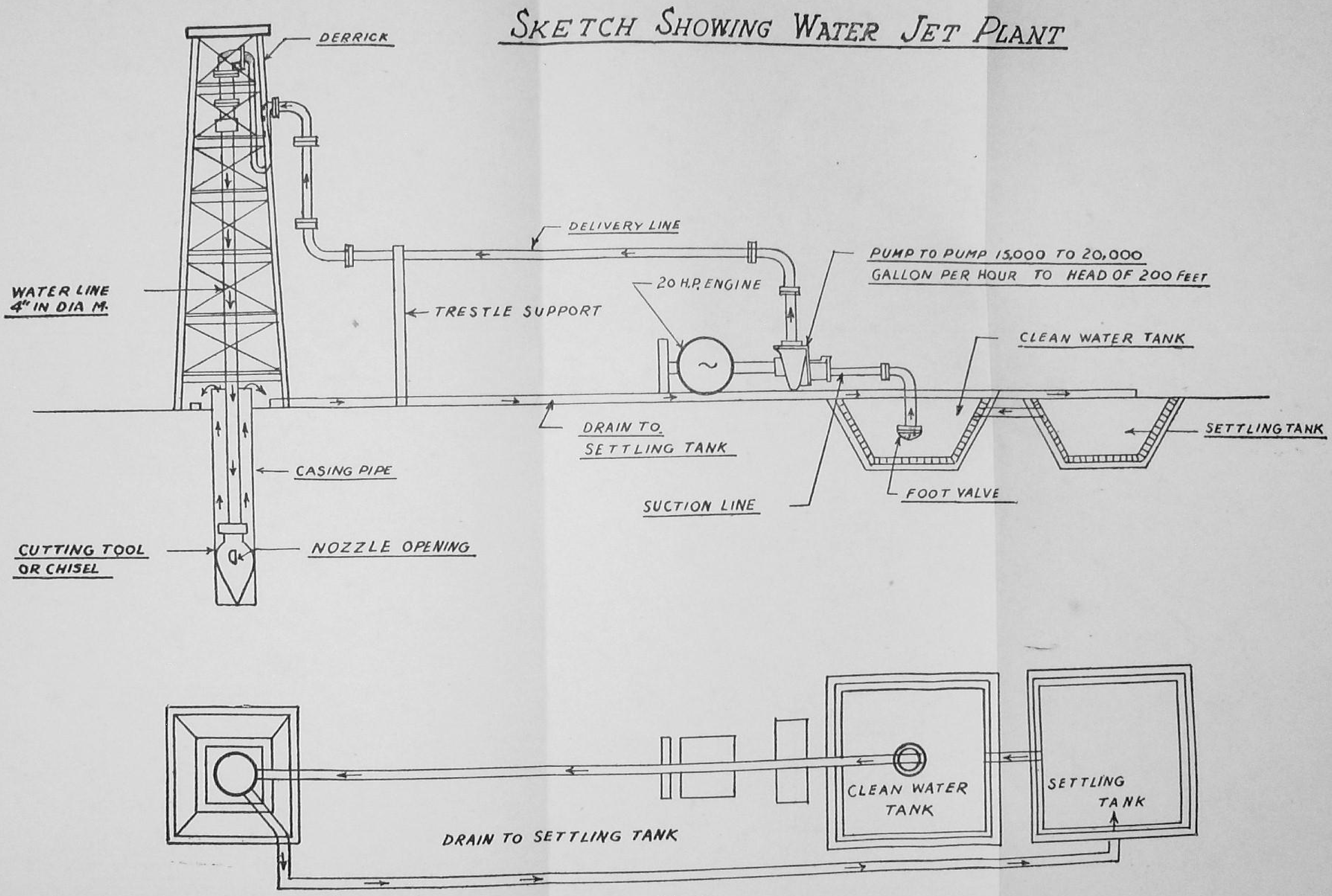

SKETCH SHOWING WATER JET PLANT
DERRICK
DELIVERY LINE
PUMP TO PUMP 15,000 TO 20,000
GALLON PER HOUR TO HEAD OF 200 FEET
20 H.P. ENGINE
WATER LINE
4" IN DIA M.
TRESTLE SUPPORT
CLEAN WATER TANK
DRAIN TO
SETTLING TANK
SETTLING TANK
CASING PIPE
SUCTION LINE
FOOT VALVE
CUTTING TOOL
OR CHISEL
NOZZLE OPENING
CLEAN WATER
TANK
SETTLING
TANK
DRAIN TO SETTLING TANK

Fig. 36.


SKETCH SHOWING WATER JET PLANT
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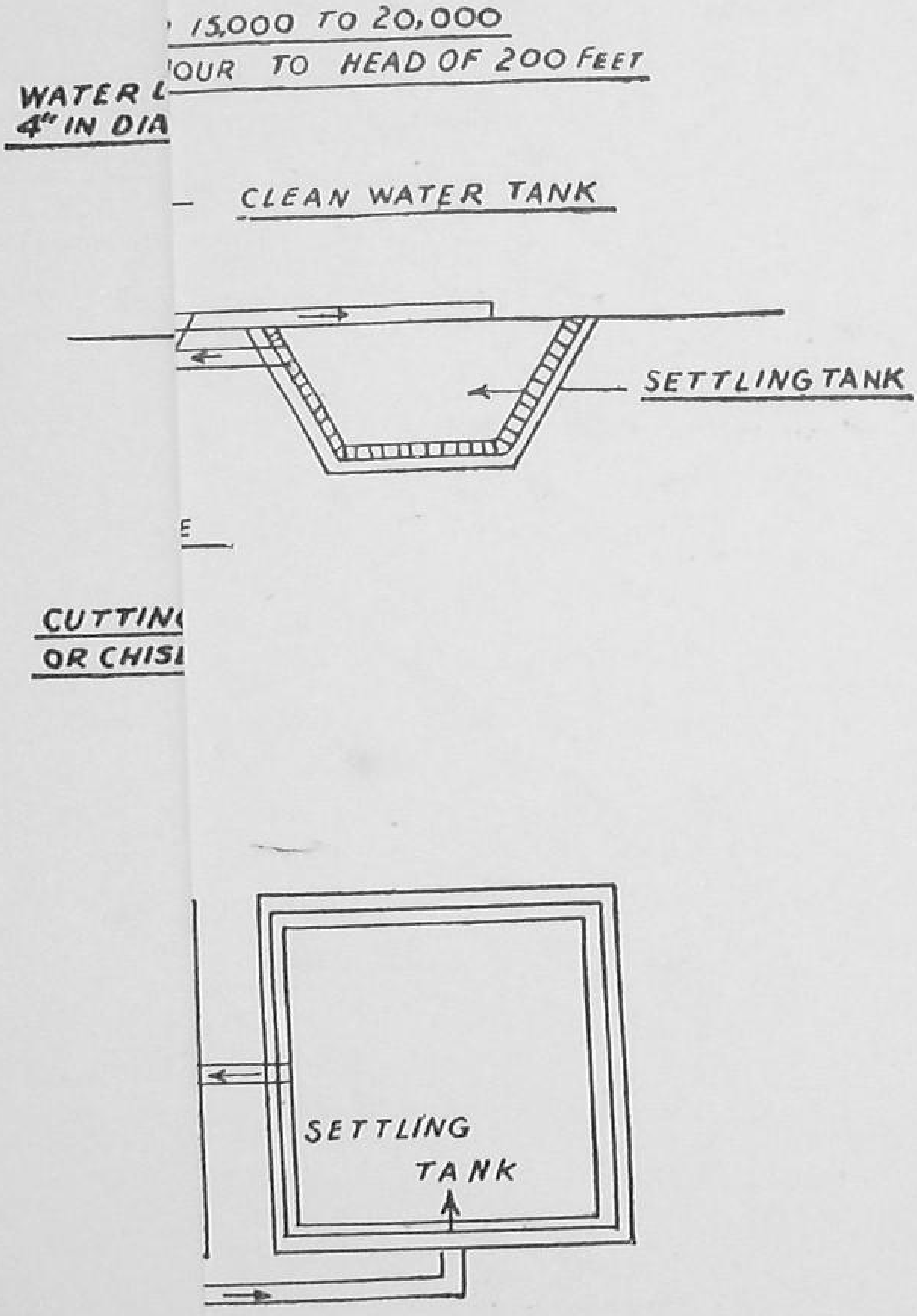
CASING PIPE
SUCTION LINE
FOOT VALVE
CUTTING TOOL
OR CHISEL
NOZZLE OPENING
CLEAN WATER
TANK
SETTLING
TANK
DRAIN TO SETTLING TANK

Fig. 36.

PLANT



15,000 TO 20,000
OUR TO HEAD OF 200 FEET
WATER L
4" IN DIA
CLEAN WATER TANK
SETTLING TANK
CUTTIN
OR CHISL
SETTLING
TANK

3. **Observations during boring.**—The material excavated during boring of a tubewell will indicate the nature of sub-soil strata at various depths. A record of these strata is very necessary as on it would depend the respective positions of perforated and plain pipes. When a water-bearing stratum is struck, water therefrom would find its way into the casing pipe. The extent to which this water rises in the pipe before work is started would give the static water pressure of the water in that aquifer provided no boring operations have been done during the night time. During the boring operations, some water is being constantly bailed out with the "sludger" whenever it is taken out to empty it of the excavated materials. The result is the lowering down of the water level. This gives the working pressure. If the inflow of water is very great, the drop would be very little and sometimes negligible. When boring is being done through clay it is obvious that very little water would rise in the casing pipe. As boring goes on, an accurate record should, therefore, be maintained of the morning and evening water levels which would indicate the static and working pressure of water in the various strata.

4. **Types of tubewells*.**—After ascertaining the water bearing strata, strainers have to be located opposite these to draw water. This is done by assembling together the whole length of the strainers and plane pipes in exactly the same order and lengths in which they are to be lowered in the bore.

Tubewells vary in designs from a strainer of perforated pipes inserted into a casing pipe of somewhat larger diameter, the annular space being then filled with gravel and the casing pipe then removed, to the Ashford patent tubewell where the strainer consists of a frame-work of bars round which copper wire is wound at high tension, giving the wire a wedge like section so that when in use, the wire presents a flat surface to the water bearing soil surrounding it with a slot between adjacent strands 1/100" in width. The description of the main types is given below:—

(i) Strainer Wells.—For satisfactory working of a strainer the straining material should not be in direct contact with the perforated tube over which it is wrapped as the area of perforation is thereby reduced. In the agricultural strainer, this condition is fulfilled by welding iron hoops and strips over the perforated tubes. The perforations are so arranged that these strips do not come over the perforated portion. The strainer sheet is wrapped and soldered over the hoops and strips. There are now different types of strainers in the market like Cooks, Tej, Phoenix, Ashford strainers etc.

*Extracted from Hand Book on Tubewells by K. D. Sanwal.

When water is pumped from the tubewell, the fine particles of sand surrounding the strainer get washed through the straining material into the tube. On account of high velocity of water into the tube this sand is carried upwards and the strainer and pipe are kept freed of its fine particles and coarse materials left behind arrange themselves around the strainer according to the size of sand grains. The surface area of the cylindrical figure enclosing the coarse particles (after being separated from the fine particles) is such that water passing through this surface has a velocity not exceeding the critical velocity of the sub-soil formation.

(ii) Gravel Wells and Shrouded Wells.—The shrouded well is a development of the strainer well in which shrouding is done around the screening so as to secure a larger percolating cylinder of coarse material and thus a greater yield of water from a particular aquifer for a specified draw-down. There are two ways in which such wells can be developed *viz.* (a) by natural gravel packing (this is known as gravel well) and (b) by shrouding or artificial gravel packing *i.e.*, adding gravel from outside for shrouding. In the case of shrouded well, the annular space between the slotted pipe and the boring pipe is filled to about 10 ft. above the top of the slotted pipe with gravel poured down from ground level. The boring tubes are then raised to about 2″ above the bottom of the slotted tube. Developing operations are then started by pumping water with the help of an air compresser. As soon as pumping is started, fine material is drawn into the coarse material and since there is a mixture of coarse and fine grains in the water bearing formation, the coarse will be held back and the fine will be pulled into the artificially packed gravel. The adding of the gravel from the outside and lifting of the boring pipe must be done gradually as the development progresses. The gravel well is developed exactly in the same manner as the shrouded well with the difference that in the former case, no adding of gravel from outside is done and advantage is taken of the presence of coarser materials in the water bearing formation itself. A section of the gravel well is shown below:—

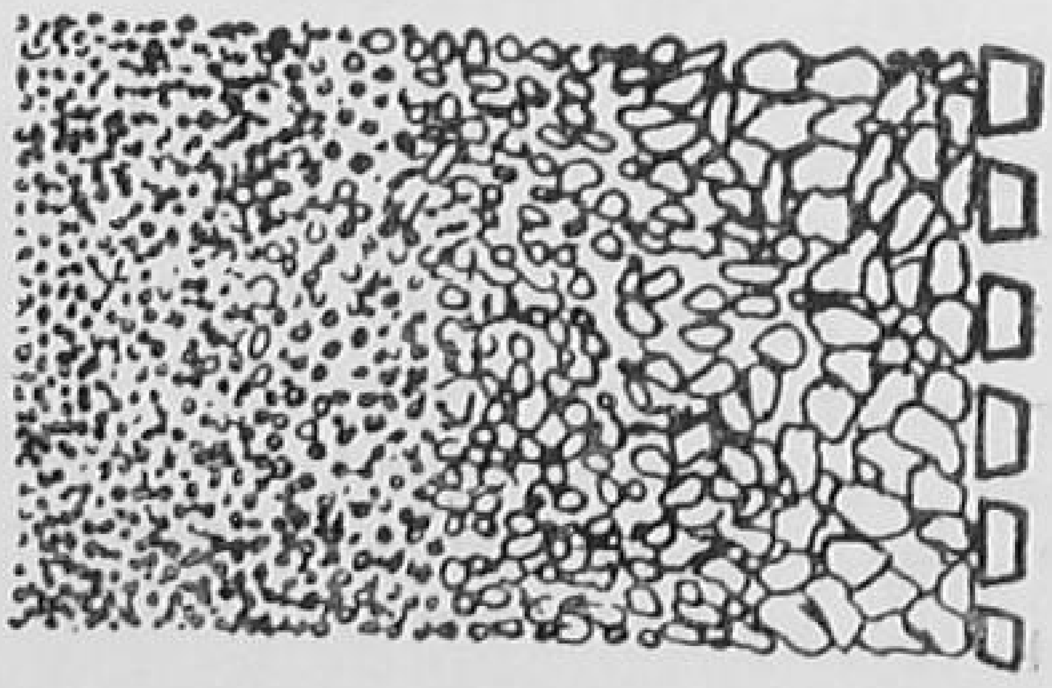


Fig. 37

5. **The Finished Tubewell.**—After the whole length of plain and perforated pipes has been suspended in position from the top, the boring pipes are jacked up and extracted one by one. As these pipes are removed, the earth and sand around the tube get loosened. They fill the hollow space around the tubewell and grip it firmly. Water from a tubewell could be lifted by directly connecting it to a pumping set which may be of the horizontal drive, vertical spindle pump or bore hole pump driven by an oil engine or an electric motor. For a centrifugal pump of horizontal drive, it is necessary to construct a sump well with its bottom about 6 to 8 ft. below the spring water level. The pump is installed as close to the spring water level as possible and if there are great variations in this level, the position of the pump has to be adjusted from time to time. When the spring water level is very low, a vertical spindle pump is sometimes used. The pump installed close to the spring water level is driven by a vertical shaft which is coupled to an electric motor or an oil engine fixed at ground level. Due to the vertical rotating spindle, the use of bearings at many places becomes necessary, and the plant on the whole becomes quite cumbersome. Its use should, therefore, be avoided as far as possible. In all modern tubewells, bore hole pumps are now commonly used. These are deep well turbine pumps with their bowls housed in the casing pipe itself and below the spring water level, thus doing away with the construction of a separate sump well. In continuation of the tubewell, a casing pipe of bigger diameter is provided at the top to a depth of 80′ to 100′ below the ground level to house the bore hole pump. The pump being constantly submerged in water needs no priming and can stand wide fluctuations in the water level resulting from large depression heads. The ordinary limits of depression heads for tubewells in this country are taken as 10 to 20 ft. The bore hole pump is, however, very costly and should be used only if it cannot be avoided. In small unit tubewells (of 10,000 to 15,000 gallons capacity) where the draw-down is not very great, it is generally possible to install with success a centrifugal pump with horizontal drive.

CHAPER VIII

THE ECONOMICS OF IRRIGATION WORKS—

THE WATER RATE

The economics of irrigation works could be examined from two considerations:

(i) The extra out-turn in terms of additional yield of food grains per unit of capital invested including the recurring expenses and its overall effect on the country's wealth.

(ii) The net revenue derived in the shape of water rate after deducting the working expenses, interest and depreciation on the capital outlay etc.

The extra out-turn of food grains due to irrigation is taken approximately as 5 mds. per acre, the cost of which varies from Rs. 60 to 80. In areas of low and uncertain rainfall this increase would be several times more. Even the most expensive method of irrigation (like drawing water from a deep well by bullocks and moat) would not cost more than Rs. 35 per acre and in that case also the cultivator would stand to gain about Rs. 25 per acre after allowing for increased incidental expenses due to increased production. It is thus obvious that purely from considerations of cash value of the extra yield of crops, practically any kind of irrigation work would be an economical proposition. On the other hand, if only the revenue return in the shape of water rate is reckoned an irrigation work need not necessarily be a paying concern. This refers to the latter aspect of the economics of Irrigation Works. The net revenue or income derived in the shape of water rate depends not only upon the extra out-turn of food grains but upon what the cultivator can pay or is willing to pay. For example, we take the case of an irrigation tank on the construction of which Government have invested, a capital of say Rs. 300 per acre of the irrigable area.

Interest on the capital outlay at 3 %	= Rs. 9-0-0
Maintenance and operational charges at Rs. 3/- per acre	= Rs. 3-0-0
Depreciation	= Negligible
TOTAL	= Rs. 12-0-0

Unless, therefore, a minimum water rate of Rs. 12/- per acre can be recovered from the cultivator, the scheme would not be able to pay for itself. If it is possible to recover more than Rs. 12/- it would be a source of income. Works that are paying, are classed as 'productive' and those which do not pay as 'unproductive'.

We now proceed to describe the respective costs of construction, maintenance and operation of the various irrigation methods and how they compare with the cash value of the extra out-turn of food grains.

FLOW IRRIGATION

Tank Irrigation.—Construction of major tanks ordinarily costs about Rs. 200 to 400 per acre of the irrigable area. It varies considerably with local conditions. Minor tanks cost somewhat more. An analysis of the economics of this work has already been given above from which it would be observed that the working expenses of an irrigation tank would range from Rs. 10 to 15 per acre. According to the rates prevalent in the country, generally the tank schemes will not be self-financing, unless a betterment fee is levied in the very beginning on the area benefiting from the scheme, thus bringing down its capital cost. In the drier parts of Rajasthan and Ajmer water losses due to percolation and seepage through sub-soil are considerable. In these areas, the cost per acre is much higher.

Canal irrigation from perennial rivers.—These schemes cost about Rs. 50 to Rs. 150 per acre.

Interest on capital outlay	= 3%
Maintenance	= 2%
Depreciation	= 1%
TOTAL	= 6%

Thus maintenance and working charges of this would cost anything between Rs. 3 to Rs. 9 per acre. Generally it would be possible to recover water rate equal to or in excess of this amount and the schemes would not only be self-financing but in many cases would be remunerative (*i.e.*, productive).

Flood Irrigation.—Where flood is diverted into suitably bunded fields for the sole purpose of wetting the land and keeping it submerged for the duration of the rains, it is generally possible to do this at 4 annas to 8 annas per acre or even less. The amount spent is little but the advantages are many and people would willingly pay for its working expenses, including the depreciation and interest on the capital outlay. This scheme also is self-financing.

LIFT IRRIGATION

Lifting water by means of Mote or Charas (Leather Bucket).—The capital cost of digging a well is about Rs. 2,000 in the alluvial plains. The leather bucket and the rope are the only movable articles. Generally, one pair of bullocks and two men can work the Mote. The lift involved is 20 to 30 ft. The discharging capacity of

the Mote varies from 1,000 to 1,500 gallons per hour, depending upon the lift and an area of one-third or one-fourth of an acre is irrigated per day. The working cost of the Mote could be analysed as below:—

Hire charges of one pair of bullocks per day	= Rs. 3/-
Labour charges for 2 men	= Rs. 2/-
TOTAL	= Rs. 5/-
∴ Cost per watering per acre	= Rs. 15/-
and cost for three waterings (for wheat irrigation)	= Rs. 45/-
Interest on the initial capital outlay at 3%	= Rs. 60/- for 5 acres or Rs. 12/- per acre
To this has to be added the maintenance and depreciation charges of the leather bucket and rope)say Rs. 10/- per year or Rs. 3/- per	= Rs. 3/-
TOTAL	Rs. 60/- per acre.

In actual practice, labour and bullocks do not cost the cultivator as much as has been assumed above. These rates apply to hired labour and hired cattle. The labourers are members of the family and the bullocks, the cultivator has to maintain in any case, as an essential part of his agricultural equipment. All this brings down the cost of irrigation to a much lower figure which is about Rs. 35 per acre. The cultivator believes that irrigation by Mote is still cheaper. He does not reckon either the cost of labour or the bullocks in the irrigation charges and thinks that irrigation by the Mote system costs him very little. When the proposition for pump irrigation (which is much cheaper, as has been analysed later on) is put forward to him, his first reaction is that he would not be in a position to bear the recurring and maintenance costs of a pump. Sometimes, he questions: What will happen to his bullocks and also the man-power available in his family when all these have been replaced by a pump and a pump driver? It is only during the recent past when pump irrigation has been introduced on an extensive scale, that the cultivators have started realizing that by conserving their bullocks, these could have a longer life, that by conserving the man-power they could do more intensive cultivation in the fields. While, therefore, irrigation by draught animals could be gradually replaced by mechanical methods, village economy is so intimately linked with bullock economy that it would not be correct to introduce pump irrigation indiscriminately.

Lifting water from a well by Plant or a Persian Wheel.—This is a much more efficient method of lifting water than the Mote even though draught animals are used in both the cases. Here the labour and the animal power can lift water to the extent of 2,500 to 3,000 gallons per hour for a lift of 20 to 30 ft. The cost of a Persian Wheel ranges from Rs. 600 to Rs. 800. Other miscellaneous expenses amount to Rs. 200. Thus the capital cost of digging of wells and

installing Persian Wheel thereon in the alluvial plains, is about Rs. 3,500. The area irrigated per day is about half an acre and above, depending upon the depth from which water is raised. The irrigation charges by this method work out to about Rs. 25 per acre. Regarding relative economics of Persian Wheel and pump irrigation, what has been stated for the Mote, holds good for the Persian Wheel also, since bullock power is used in both the cases.

Pumps.—In contrast to the methods of lifting water as described above, pumping machinery of high efficiency is now commonly used for irrigation purposes. By this method, not only irrigation becomes more efficient, but costs of lifting water are greatly reduced by obtaining the necessary energy for pumping from coal, gasoline, crude oil and electricity, rather than using man or animal power. To illustrate:—

One K.W. hour (being one unit of electrical energy) may be purchased at -/2/- as. One unit is roughly equal to 4/3 Horse Power Hour and hence one H.P. hour will cost -/1/6 as. A strong healthy man can generate in one hour about one-eight Horse Power work. A man these days earns about Rs. 1/4/- to Rs. 1/8/- a day or 3 as. per hour (taking 7 hours as the working period). Thus to generate one Horse Power hour by using man-power, would cost Rs. 1/8/- against -/1/6 as by using electricity. In addition, the efficiency of the pumping operation by using modern pumping methods, is considerably increased. In the case of man or animal operated lifting devices, the efficiency is from 10 to 20 per cent. The efficiency of the machine operated pumping plant varies from 25 to 70 per cent. and averages at 50 per cent.

A well of normal yield has water supply enough to cope up with the draught by a 3 Horse Power pump. Details of a typical electrically driven pump, are given below:—

The pump worked for $2\frac{1}{2}$ months and 1600 units of electricity were consumed.

Area irrigated = 20 acres during the period.

[1600 units = 1600 × 4/3 horse power hour.

Therefore the number of hours the pump worked during the year is 1600 × 4/3 × 1/3 = 700 approximately.

Assuming the discharge of the pump to be 5000 gallons per hour, quantity pumped during 700 hours = 35,000,000 gallons = 160 acre inches = 8" per acre. This is the water required for rabi irrigation.]

Cost of machine (3 Horse Power Electric Pumps) . .	= Rs. 1,400
Cost of accessories and installations	= Rs. 1,100
TOTAL . .	= Rs. 2,500

Interest per year at 3%	= Rs. 75
Cost of 1600 units of electricity at 2 annas per unit . .	= Rs. 200
Depreciation (assuming 7000 working hours as life of the pump)	= Rs. 140
TOTAL . .	= Rs. 415
Therefore, cost per acre	= Rs. 20 to 21

The details for a typical 8 to 10 H.P. oil engine installation are also given below:—

Cost of pump	= Rs. 820
Cost of engine	= Rs. 2,000
Installation	= Rs. 1,000
TOTAL . . .	= Rs. 3,820
Interest at 3%	= Rs. 115
Depreciation (taking 10,000 hours as the life of the pump)	= Rs. 170
TOTAL . .	= Rs. 285

The engine worked for 8 hours a day. It consumed 8 gallons of oil (powerin) and ½ gallon mobile oil for 16 hours working.

Total oil required during 2½ months	= 300 gallons
and total mobile oil consumed	= 9 "
Cost of 300 gallons of powerin @ Rs. 1/12/- per gallon	= Rs. 525
Cost of mobile oil @ Rs. 8/- per gallon	= Rs. 72
Half driver for 4 months	= Rs. 140
TOTAL . .	= Rs. 737
TOTAL CHARGES	= Rs. 1,022 for 50 acres.
or cost of irrigation per acre	= Rs. 20/-

Tubewells

Tubewell irrigation has been practised most extensively in the State of Uttar Pradesh and has recently been introduced in Punjab also. Each tubewell unit of one cusec, used to cost Rs. 11,300. The cost rose steeply after 1942-43 and each tubewell project then cost Rs. 27,000. The rise continued and the cost at present, is Rs. 37,000 and above for 1·5 cusec tubewell. It costs Rs. 39,000 with a Diesel engine pumping set.

Working Expenses

Interest @ 3%	= Rs. 1,170/-
Depreciationon the cost of equipment and machinery only taking its life as 15 years	= Rs. 1,200
Establishment (overseers, operators, pump attandant etc.)	= Rs. 1,750
With 60 per cent efficiency, the electrical energy consumed per foot of pumping head for one cusec would be ·14 K.W. per hour.	
Total head will be about 40 ft.	
Therefore units consumed	= 5.6 K.W. hour or 8.4 units for 1.5 cusec TubeWell.
Therefore energy consumed during the year	= 4000×8.4 = 33600 units
(Assuming 4,000 working hours per year.)	
Cast of energy @ 2 As. a unit	= Rs. 4,200

TOTAL . .	= Rs. 8,320/-
Therefore cost of 4000 × 1.5 = 6,000 cusec hours . .	= Rs. 8,320
or the cost per acre inch = 8,320/6,000	= 1.4
(One cusec hour—1 acre inch)	
i.e. the rate per acre inch	= Rs. 1/7/-

For wheat, the depth of water is taken as 8″ to 9″ and the cost of irrigation would thus be Rs. 12 to 14 per acre.

On tubewells, it has been possible to arrange volumetric sale of water; hence all states have a basic rate for a volume of water delivered at the tubewells. Punjab has been assessing 11,000 gallons of water to a rupee, while Uttar Pradesh proposes charging a rupee for 16,000 gallons. On the basis of cheaper assessment planned by Uttar Pradesh Irrigation, the rate works out to Rs. 11 only per acre for wheat.

CHAPTER IX

MINOR *VS.* MAJOR IRRIGATION WORKS—MAINTENANCE OF MINOR WORKS

"Questions are often raised about (i) the relative place assigned to major and minor irrigation schemes in the Plan and (ii) the economics of such schemes. There can be no conflict between major and minor schemes. There are parts of the country in which scope exists for large projects and there are others in which only smaller projects are possible. Each area should be served by the kind of schemes for which it offers facilities. Large and small projects are thus complementary and not competitive. The relative advantages and disadvantages of minor and major schemes may be summarised as follows—Minor schemes are advantageous in that (i) the initial outlay involved is small, (ii) they can be executed quickly and yield quick results, (iii) they generally require no special assistance by way of foreign personnel or equipment and (iv) local resources can be easily mobilized for their execution. The disadvantages on the other hand are—their cost of maintenance, their relatively short life and the limited 'protection' they give"*.

The real problem with the minor irrigation works is their eventual maintenance and management. Experience in this country has shown that minor irrigation works can be maintained only if the beneficiaries undertake the obligation for this. At the same time, where there are laws imposing such obligations, it has not been possible to enforce them strictly. As remarked by Shri V. T. Krishnamachari, Deputy Chairman, Planning Commission in the 2nd Development Commissioners' Conference, "We have been spending crores of rupees on minor irrigation works during the past several years but the net area actually irrigated by such works, has been decreasing". If this problem is not effectively tackled, he continued, "We shall only be increasing the numbers of derelict tanks in the country". For efficient operation of these works, it is felt that, to start with, all minor works (except wells and small scale tubewells), whether newly constructed or repaired by Government, should be maintained and managed by the State concerned till irrigation from these works has fully developed. For privately

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*Page 364 of the FIVE YEAR PLAN.

owned tanks, which are repaired at State expense, necessary legislation* could be enacted to enable Government to requisition such works for certain period without the need of permanently acquiring them. After irrigation has fully developed:—

(i) The newly constructed works could be handed over to a Panchayat or the Village Council, which should be made responsible for their proper up-keep and maintenance. The beneficiaries should contribute regularly to a non-lapsable irrigation fund and if, at any stage, it is found that the works have not been properly maintained, necessary repairs could be carried out by the State Government and costs recovered from the funds thus collected. A responsible irrigation officer should be required to inspect all the minor works once a year, who should clearly specify what repairs are needed to keep the work in good order. For making the scheme self-financing, wherever this is found practicable, land revenue of the area declared as irrigable, could be suitably enhanced at the time of settlement. Before settlement, the beneficiaries should be required to pay to Government an equal amount of tax in the form of water rate. This would be in addition to the irrigation fund referred to above. In the State of Bilaspur (Simla Hills) the Panchayats have powers to order an *ad hoc* increase in the Land Revenue, whenever fresh area is brought under irrigation. This is regularized during the time of subsequent settlement. A similar Minor Irrigation Act exists in the Ganjam district of Orissa and Hyderabad and probably in parts of Madras Presidency where the moment any land comes under irrigation, from any source whatsoever, it is automatically assessed to water-rate. While it is necessary to decentralize the control of the minor works and leave it to the management of the beneficiaries as far as possible, it should not be done at the risk of allowing these works to go into a state of disrepairs, as has been the fate of several minor works all over the country.

(ii) The requisitioned works could be released from requisition provided the original owners or the beneficiaries agree to reimburse the State Government, the amount spent on their repairs. Reimbursement could be effected in instalments. The management and maintenance of these works should also be entrusted to the beneficiaries as for new works.

*Irrigation Amendment Act 1948 in Madhya Pradesh.

The maintenance of minor irrigation works as indicated above raises an important issue *i.e.*, how far it will have the people's sanction behind it and whether by exercising the control in this manner, we shall not be encroaching upon the right of cultivators to maintain and manage works of which essentially they are the authors. To some extent this disadvantage would be there and it may not perhaps wholly conform to the spirit of a people's programme, but if a broader view is taken such a course seems unavoidable in wider national interests. This would only be a temporary phase and as the people become more and more educated, more irrigation-minded and develop a higher sense of co-operation, they are sure to realize that such steps are necessary to safeguard their interests, to make their irrigation system more efficient and more permanent, and to improve the general economic condition of the country as a whole.

A case in example is the "Compulsory Irrigation Act" in the State of Madhya Pradesh. The Act visualizes compulsory assessment, from year to year, of land once declared by a Gazette Notification as irrigable from an irrigation work, whether such land is in need of irrigation or not or whether it is actually irrigated or not. This Act has so far not come into force, but once it does (as it should very soon) it would impose a certain amount of obligation on he cultivator to make the best use of the irrigation work, which left to himself he would not ordinarily do. In the state of Madras, the Minor Irrigation Act on somewhat similar lines as the Compulsory Irrigation Act of Madhya Pradesh has been in force for the past several years and it has the full sanction and backing of the cultivators in that State. The Minor Irrigation Act of Mysore is still more interesting. All the irrigation tanks, big and small, are the property of Government. The Atchkatdars (*i.e.*, the beneficiaries) are responsible for the maintenance of earth-work in the bund and are also required to contribute regularly to a fund, known as 'Irrigation Cess Fund' (I.C.F.) at the rate of one anna per rupee of the land revenue in which is merged the irrigation water rate also. The amount available under the I.C.F., which is rather inadequate, is utilised for maintaining the masonry works in order. This item is a responsibility of the Government. When due to continuous neglect on the part of beneficiaries, a tank breaches, its restoration is carried out under the provisions of the Minor Irrigation Act. According to this Act, when a tank has to be restored, an estimate is prepared for its repairs by the Irrigation Officer and the amount notified to the beneficiaries. If they agree to share one-third of the cost, Government meet the remaining two-third cost and the work is carried out. The one-third cost is usually recovered from the cultivator as a revenue demand, though recently people have been coming forward to give their contribution either in cash or in kind at the time of actual

restoration. Here also, inspite of most elaborate provisions in the Minor Irrigation Act, the tanks continue to go out of repairs. The main reason for this appears to be the failure on the part of the cultivators to discharge their obligation properly. Even though Government may decide to carry out the repairs at the Ryots cost, funds are required in the initial stages. It is not always possible to arrange for these funds and the tanks go out of repairs. The remedy to this would appear to be either to increase the contribution to the I.C.F. or the creation of a non-lapsable irrigation fund by the State as visualised in the Five Year Plan (page 361) so that shortage of funds does not come in the way of repairs and restoration work.

In brief, wherever irrigation is not fully developed, it would be unavoidable, for its proper development, to impose some kind of obligation on the cultivator to maintain his irrigation works in proper condition and to make the best use of available water.

CHAPTER X

THE ENGINEER'S ROLE IN THE COMMUNITY PROJECTS

We in India, have 49 million acres of land under irrigation, of which approximately 22 million acres is irrigated by the State-owned tanks and canals. The rest is irrigated by small irrigation works owned privately. Under State irrigation works (both major and minor) the various State Governments have already drawn ambitious schemes on the lines visualised in the Five Year Plan and whatever extra area is brought under irrigation as a result of these works would go to increase the 22 million acres mentioned above. We, in the Community Projects, have entirely a different duty. The 27 million acres of land that is under irrigation to-day from the privately owned works is a result of thousands of years of people's efforts, so far un-aided and un-guided by any engineer. The innumerable wells in the Northern India, the Dongs in Assam, the Koohls of the Northern Hills and the numerous minor irrigation tanks dotting the whole of the Southern and Central India—these are the works initiated and built by the people in their constant struggle against famines, draughts and vagaries of monsoons—all practically without any technical assistance. It is not true that all these works have sprung up in the course of a single generation. It is also not true that all what the people thought and planned came out true or what they built functioned successfully. In fact, failures, would claim a much larger percentage of their attempts. But all the same, none can deny that these works, whatever has survived of them, have been extremely useful. Irrigation by the peasant is and has been the very life of Indian agriculture. It has grown, through ages, as a part of his rural surroundings, in much the same way as his mud-huts, his farms, his customs and his practices have grown surviving the ravages of time. His works, though crude in construction, represent a dynamic process—the slow but nevertheless sure transformation of the village life. When we construct and give a big irrigation work, to the cultivator, he gratefully acknowledges its blessings. But when we do not, he does not feel helpless. He constructs a tank which may not collect any water or would possibly be washed away. He will construct another and probably a third one so long as he does not succeed. He will learn by trial and error. Similarly, he would dig a well which would probably have no water. He will construct a 'dong'. The 'dong' will become a river and water will not flow into the field. He would repeat all these operations till he sees water actually flowing into the fields. It is by this gradual process that the net-work of the minor irrigation system has been built up to irrigate the 27 million acres of land. In these Community Projects

could we not guide and advise the cultivator and plan his irrigation work to some extent without attempting to bring it wholly to the pattern of State-owned canal system. The State-owned works are planned and designed after very detailed investigations and surveys and conform to the latest principles of engineering practice. The chances of risks are reduced to a minimum in these works. On the other hand, the cultivator's works are planned and initiated by him. By our association with his activities, if we could cut down the failures say from 4 to 2, it would make a very big difference. Failure of works cannot be eliminated much less completely. The important point is that the engineer's association with the villagers works should not damp the tempo of his activities and must keep pace with the moving force in Indian agriculture. All this will need a certain amount of reorientation in the attitude and ideas of our engineers and other technical personnel. So far they have been adapted to design and construct State-managed works only, which must necessarily conform to the standard blue-print, the type designs and the approved specifications. The cultivators works do not and cannot come up to this standard for obvious reasons. A deviation from the approved practices in these cases is thus unavoidable and their designs and construction must inevitably entail a certain amount of risk. Our engineers are not accustomed to take such risks in the planning and investigation of an irrigation scheme or to make deviation from the standard practices. It is obvious that detailed planning and strict adherance to blue-prints would defeat the purpose of the minor irrigation works visualized in the Community Projects. What is wanted is how best we could assist the cultivator in constructing his own works a shade better than what he has been accustomed so far: how by aligning an irrigation channel along the water-shed, water could be made to flow in most of the cultivable land by gravity; how by constructing a masonry diversion weir, frequent bunding of the stream by bamboos, brushwood and earth, could be avoided, how by constructing a simple channel head, a controlled supply could be let out into the irrigation channel instead of allowing it to get choked with the full flood water and so on. "We have to adapt ourselves to the new conception of the Welfare State. We should approach our task in a new spirit. We should give up the idea that we are here to force our programmes and plans on the villagers. It is for the villagers to tell us what they want and for us to assist them. We can advise them. We can organise their efforts. We can give such financial assistance as may be needed. But the initiative in the selection of plans must be that of the villagers and the Village Level Workers and the Extension staff should be their guides and friends. We shall all find as our work progresses that villagers have more to teach us than what we have to teach them".*

*Extracted from Shri V.T. Krishnamachari's Remarks in the 2nd Development Commissioner's Conference from 16th to 19th April, 1953.

CHAPTER XI

CONCLUDING REMARKS

While the pages of this manual are being written, the country is on the eve of embarking on a much bigger experiment than the Community Projects and this is the National Extension Service, wherein it has been visualized that approximately one-fourth of the country will be covered up by the Extension Service during the coming three years and the whole country during a period of about 10 years. The degree of success achieved in the implementation of Minor Irrigation Programme in the Community Projects would be a measure of what we would possibly accomplish in the National Extension Service. In many parts of the country, the physical barriers may prove to be too great to be easily overcome and the irrigation projects there, may have to be dropped or slowed down, being unproductive. In others lack of technical personnel may hamper execution of the irrigation programme and still in another part, the land problem may be so serious that irrigation may have little or no appeal to the landless cultivator. The indirect benefits of irrigation are so great that time has come when we must sink money on irrigation works even in areas less favourably placed by Nature or where due to land and other economic problems, the cultivator is in no mood to pay the cost price of water. Wherever, there is deficiency of technical staff we could start short term training and orientation centres and turn out overseers and engineers to work on the design and construction of minor irrigation projects. Where there is the land problem, introduction of land reforms could be speeded up. In short, we must effectively and systematically smash all the barriers and have the irrigation programme through since without it no development programme could be complete. We have talked of the "human problem" and rightly emphasized the need for a "human approach" but we must not lose sight of the fact that human virtues cannot grow and thrive in vacuum. We must have concrete works coming on the surface and concrete areas brought freshly under irrigation even if the human approach has to be sacrificed to a certain extent. Later on, as irrigation develops, as more food is produced in the country, as the era of plenty dawns, the human qualities would themselves spring out of the hearts of a prosperous and contented people.

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